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Albuquerque District

RIO GRANDE FLOODWAY
Truth or Consequences Unit, NM



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CUCHILLO NEGRO DAM

FOUNDATION REPORT, Volume I
Report, Photographs, and Appendix A

CONSTRUCTION FOUNDATION REPORTS
ER 1110-1-1801

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Cuchillo Negro Dam was completed in July 1991. The project, which is a dry, flood control only reservoir, consists of a main dam and an auxiliary spillway. The dam consists of a 750-foot-long earth embankment section and a 590-foot-long roller compacted concrete (RCC) section. The crest of the earth embankment section is 21 feet wide, and it is 25 feet wide for the RCC section. The dam has an overflow spillway and rises 134 feet above the channel bottom. The auxiliary spillway is an RCC lined spillway with a conventional concrete Ogee. It is 680 feet wide and extends from a point 260 feet from the right abutment of the RCC dam in a southeasterly direction. Numerous unanticipated geologic conditions were encountered during excavation of the foundation trench. This led to design changes and modifications to the contract.

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CUCHILLO NEGRO DAM
RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, NM

FOUNDATION REPORT
DECEMBER 1992

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RIO GRANDE FLOODWAY, T. OR C. UNIT, N.M.
FOUNDATION REPORT
RCC DAM, EMBANKMENT DAM, RCC SPILLWAY, AND OUTLET WORKS

I - INTRODUCTION

1.01 Location and Description of the Project. The Cuchillo Negro Dam is located in Sierra County, New Mexico on Cuchillo Negro Creek, a tributary of the Rio Grande River, approximately 8½ miles northwest of the city of Truth or Consequences and 2½ miles downstream of the village of Cuchillo. The drainage area of the project is approximately 364 square miles. Table 1-1 lists the pertinent data concerning the reservoir. Plate No. A-1 in Appendix A is a site and vicinity map of the project area.

Table 1-1 Reservoir Pertinent Data

	Maximum Water Surface Elevation, Feet NGVD	Capacity Acre Feet	Area Acres
Probable Maximum Flood	4,739.4	20,500	520
One Percent Chance (Design) Flood	4,721.1	13,500	385
Flood Control Storage		7,500	
100 Year Sediment Accumulation		6,000	

1.02 Construction Authority. The Water Resources Development Act of 1986 modified the previous authorization as contained in the Flood Control Acts of 1948 and 1950. Section 871 of the Water Resources Development Act of 1986 states:

> "Subject to section 903(b) of this Act, the project for flood protection for the Rio Grande Floodway, Truth or Consequences Unit, New Mexico, authorized by the Flood Control Acts of 1948 and 1950, is modified to provide that the Secretary [of the Army] is authorized to construct a flood control dam on Cuchillo Negro Creek, a tributary of the Rio Grande, in lieu of the authorized floodway."

1.03 Purpose. The purpose of this report is to compile and present all pertinent information on foundation conditions encountered during construction for future reference as required by ER 1110-1-1801. Included are discussions of the general geology, foundation explorations, excavation procedures, character of the foundation, foundation preparation and treatment, and possible future problems. Also included are pertinent tables, figures, plates, and photographs.

1.04 Location and Description of Structures. Cuchillo Negro Dam extends, from its northernmost end, generally in a southwesterly direction for 1,340 feet across Cuchillo Negro Creek in sections 35 and 36, T.12S., R.4W., New Mexico Principal Meridian. It consists of an earth embankment section on the left abutment and a roller compacted concrete (RCC) section across the channel, tying into bedrock on the right abutment. The high level outlet works intersects the dam axis at STA 1+88.00C. The low level outlet works intersects the dam axis at STA 2+63.00C. The center of the dam spillway intersects the dam axis at STA 3+12.00C. The auxiliary spillway is 680 feet long, extending southeast from a point 260 feet from the southernmost end of the dam. The low level outlet works conduit is fed by an ungated, ported intake tower with 20 ports. Each port is 24 inches long and 6 inches high. The high level outlet works is ungated with a trash rack at the portal. The stilling basin at the base of the dam is 120 feet wide and 57 feet long. Table 1-2 lists the pertinent data concerning the structures. Plate No. A-2 in Appendix A is a project layout plan which illustrates the location of these structures. Plate No. A-3 in Appendix A is an aerial view of the completed project.

1.05 Contractor Personnel. The embankment, auxiliary spillway, and all appurtenant structures were constructed by the prime contractor, PCL Civil Constructors, Inc., 67 East Weldon Ave., Suite 367, Phoenix, Arizona, 85012-2044. Surveying services were subcontracted to Greene-Mecham Engineering, Inc., 7776 Point Pkwy. West, #138, Phoenix, Arizona, 85044. Drilling, blasting, and rock bolt installation services were subcontracted to McCaw's Drilling (USA), Inc., 1645 Court Pl., Suite 315, Denver, Colorado, 80202. Materials quality control was subcontracted to Western Technologies, 8305 Washington Pl. N.E., Albuquerque, New Mexico, 87113. Table 1-3 lists key Contractor personnel.

1.06 Design and Construction Personnel. The Albuquerque District, U.S. Army Corps of Engineers (CESWA) was responsible for the development of the plans and specifications for the project and performed contract administration and construction oversight. The project was designed by Boyle Engineering Corporation, 6400 Uptown Blvd. N.E., Albuquerque, New Mexico, 87110. The Walla Walla District, U.S. Army Corps of Engineers (CENPW) provided the concrete mix designs and specifications. Title II materials QA services were provided by Sergent, Hauskins, & Beckwith Consulting Engineers, 4700 Lincoln Rd. N.E., Albuquerque, New Mexico, 87109. The City of Truth or Consequences, New Mexico was the local sponsor of the project. Table 1-4 lists key design and construction personnel.

Table 1-2 Structures Pertinent Data

Structure	Crest Elev. Feet NGVD	Crest Width Feet	Crest Length Feet	Freeboard Feet	Discharge CFS
RCC Dam	4,742.4	25	590	3	N/A
Dam Spillway	4,721.1	Ogee	120	N/A	35,000
Embankment Dam	4,744.4*	21	750	5	N/A
Auxiliary Spillway	4,721.1	Ogee	680	N/A	199,150

Structure	Invert Elev. Feet NGVD	Conduit Length Feet	Conduit Type
Low Level Outlet Works	4,618.0	87	5-foot-diameter circular
High Level Outlet Works	4,681.0	57	7'2" X 10'0" rectangular

* includes 2.0 feet of added freeboard

Table 1-3 Key Contractor Personnel

Thomas O'Donnell (PCL)	Project Engineer
Kevin Joe (McCaw's)	Blast Superintendent
Michael Beel (PCL)	Safety Officer
Patrick Barthal (PCL)	QC Manager
Seth Smith (PCL)	Concrete Superintendent
Willis McIntosh (PCL)	Excavation Superintendent

Table 1-4 Key Design and Construction Personnel.

LTC Steven Dougan (CESWA)	District Engineer
Kristopher Schafer (CESWA)	Project Manager
David Wright (CESWA)	Ch, Geotech Branch
Dwayne Lillard (CESWA)	Ch, Soils Design & Dam Safety Section
Jim Medina (CESWA)	Civil Engineer (Soils Design)
Christopher DeWitt (CESWA)	Project Geologist
Steven Tatro (CENPW)	Materials Engineer (RCC Design)
Fred Burns (Boyle)	Design Engineer
Donald Pfister (CESWA)	Area Engineer (Construction)
Wiley Isom III (CESWA)	Resident Engineer (11/89 - 5/91)
Michael Beyer (CESWA)	Resident Engineer (5/91 - completion)
J. Wilkens & M. Mizani (CESWA)	Construction Inspectors

1.07 Contract Costs. The original bid value of the contract was $8,014,285.00. Contract Modifications P00002 through P00056 added $2,043,591.51 to the cost of the contract. In addition, variations in estimated quantities added $736,363.05 to the contract. Final cost of the contract was $10,794,239.56. Table 1-5 lists the contract bid items, and provides the estimated and actual quantities for each item.

Table 1-5 Contract Bid Items, Estimated vs. Actual Quantities

Item No.	Unit	Description	Est. Quant.	Actual Quant.
1	Job	Dewatering & Care of Water	1	1
2	CY	Excavation, Common	589,000	367,134
3	CY	Excavation, Rock	60,000	81,694
4	CY	Embankment Fill, Random	55,000	34,381
5	CY	Embankment Fill, Semi-Impervious	18,000	18,258
6	CY	Filter Material	4,800	1,386
7*	SY	Gravel Surfacing	47,000	0
8	CY	Dumped Rock - 12"	1,700	1,845
9	CY	Dumped Rock - 18"	2,800	2,690
10*	CY	Bedding Material	1,800	1,820
11a	CY	Excav, Dental - First 500 CY	500	500
11b	CY	Excav, Dental - Over 500 CY	500	4,174
12	SY	Compacted Subgrade	47,000	0
13	SY	Rock Foundation Contact Treatment	1,100	4,300
14	HRS	Add'l Roller Passes for Cmpctn	100	36
15a	LF	Rock Anchor Bolts - First 500 LF	500	500
15b	LF	Rock Anchor Bolts - Over 500 LF	500	397
16a	SF	Chain Link Fabric - First 1,000 SF	1,000	1,000
16b	SF	Chain Link Fabric - Over 1,000 SF	1,000	8,720
17**	CWT	Portland Cement, Type II	147,000	188,348
18**	CWT	Flyash (Pozzolan) Type F	48,900	110,712
19	GAL	Water Reducing/Retarding Admixture	14,000	15,238
20	CY	Roller Compacted Concrete, Dam	72,000	75,501
21	CY	Roller Compacted Concrete, Spillway	31,700	21,653
22	CY	Concrete, Outlet Works	837	675
23	CY	Concrete, Stilling Basin	188	122
24	CY	Concrete, Parapet Walls	68	71
25	CY	Concrete, Training Walls	86	151
26	SY	Vertical Facing System	5,250	5,807
27	CY	Concrete, Ogee Crests	6,900	6,160
28	CY	Concrete, Encasement	200	264
29	LB	Reinforcing Steel	495,000	632,943
30	SY	Rock Foundation Prep for RCC & Conv	4,500	16,701
31	LF	Reinforced Concrete Pipe - 60"	87	87
32	LF	Waterstops	40	28
33	LB	Steel Pipe Handrail	3,320	4,525
34	LB	Steel Trash Rack	15,400	8,567
35	LF	Guard Rail	2,120	1,350

Table 1-5 cont'd

36	CY	Required Waste Rock Fill	8,400	8,519
37	CY	Required Waste Fill	174,000	40,884
38	JOB	Permanent Project Sign	1	1
39	JOB	Precise Alignment Monuments	1	1
40	JOB	RCC Test Section	1	1
41	LF	PVC Pipe & Fittings	1,975	1,020
42	JOB	Access Road Maintenance	1	1
43	JOB	Bidding Bond Premium	1	1
44	SY	Presplitting	6,230	6,230
45	CY	19"-Thick Concrete on Top of Dam	370	515

* Final quantities are zero due to the elimination of the maintenance road.
** Final quantities are the result of increased RCC volume and CENPW's changes to the mix designs.

II - GEOLOGY

2.01 Geologic Setting. The Cuchillo Negro Dam site is located on the north end of the Mud Springs Prong of the Caballo Uplift, a horst block tectonic feature within the Rio Grande Depression. The Rio Grande Depression is a narrow, north-northwesterly to north-northeasterly trending active rift zone that is an extension of the Basin and Range Physiographic Province. The rift zone depression separates the Colorado Plateau Province to the west from the Great Plains Province to the east. The Mud Springs Prong is bounded to the north by the Engle Basin, and to the south by the Palomas Basin. The Mud Springs Prong, Caballo Uplift, and Engle and Palomas Basins are tectonic features within the Rio Grande Rift. Plate No. A-4 in Appendix A is a tectonic diagram of the Rio Grande Depression which illustrates the location of these features.

2.02 Physiography. The Mud Springs Mountain Range, approximately 5.5 miles long and 1.5 miles wide, is the topographic expression of the Mud Springs Prong. It is a relatively low, narrow range with a maximum relief of 1,400 feet above the floor of Cuchillo Negro Creek. The north end of the Mud Springs Mountains is just now being exposed by erosion. Cuchillo Negro Creek, in the project area, flows southeasterly through a narrow canyon near the northernmost end of the Mud Springs Mountains. It is an ephemeral stream with headwaters located approximately 28 miles northwest of the site at the north end of the Black Range Mountains. Just upstream of the site, the Cuchillo Negro Creek valley is 2,000 feet wide, but it narrows to 50 feet to 75 feet wide at the damsite. Here, Cuchillo Negro Creek flows through a steep-walled canyon 120 feet deep. The creek and its tributaries exhibit a dendritic to trellis drainage pattern.

2.03 Regional Geology. Plate No. A-5 in Appendix A is a regional geologic map that illustrates the structural and stratigraphic features described in Paragraphs 2.03a and 2.03b and subparagraphs.

a. Structural Geology. The Mud Springs Prong, considered an intra-rift horst block, is a northwest-trending homocline with bedding dipping to the northeast, generally at 20° to 25°.

(1) Folding. Southeast of the project, there are some dips that have been measured at angles of up to near vertical. The entire homocline plunges gently to the north and exhibits secondary, very gentle folding parallel to the axis.

(2) Faulting. A northwest-trending, range-bounding fault, represented by the steep escarpment of the Mud Springs Mountains, either branches into, or turns to become, a northeast-trending fault that extends through the auxiliary spillway and the field office site. To the southeast of the site, this fault joins the northeast-trending Hot Springs fault located east of the town of Truth or Consequences. Numerous smaller, en echelon faults are located throughout the region. The majority of faults are normal, strike N20°W to N20°E, and are the result of continued seismic activity related to rifting.

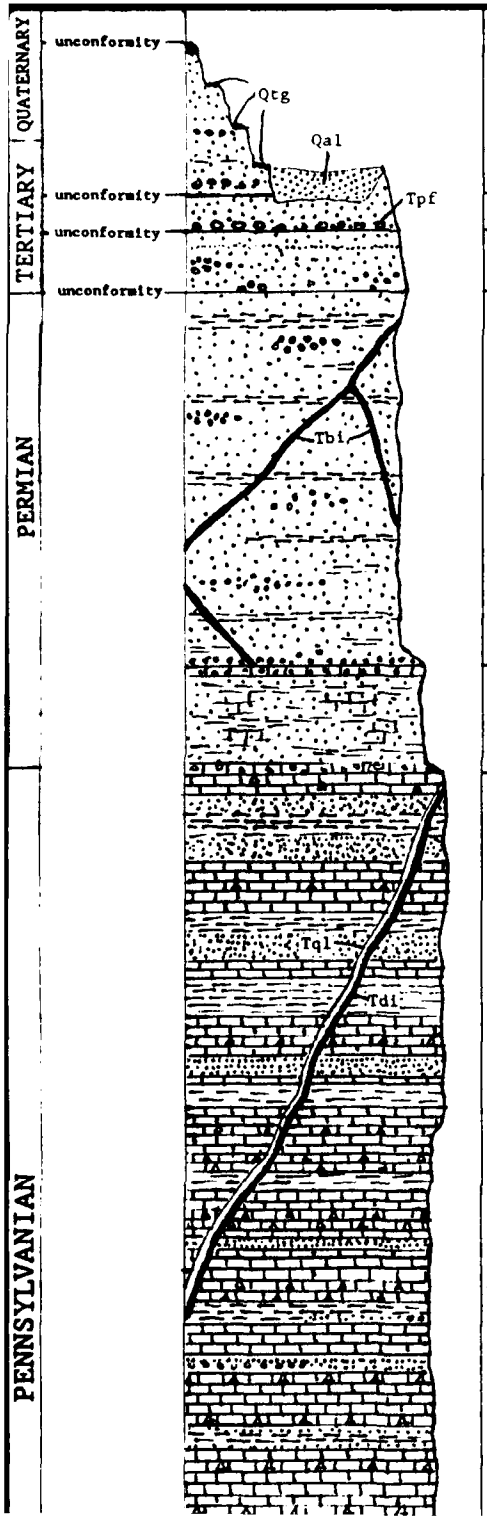
b. Stratigraphy. Rocks exposed in the region range in age from Precambrian through Quaternary. A Precambrian complex of quartzite, schist, and gneiss is exposed at the base of the Mud Springs Mountains southeast of the site. The majority of exposures, however, consist of Paleozoic marine sediments ranging in age from Cambrian to Permian, and Cenozoic continental sediments. The only Paleozoic rocks actually exposed at the damsite are Pennsylvanian limestone and shale. A thick section of generally flat-lying Miocene and Pliocene continental sediments unconformably overlies the Paleozoic section on the flanks of the Mud Springs Mountains. Minor outcrops of Cretaceous, Tertiary, and Quaternary intrusive and extrusive igneous rocks are also exposed in the Mud Springs Mountains. A regional stratigraphic column is shown in Figure 2-1.

c. Seismicity. The Rio Grande Depression rift zone is the most seismically active area in New Mexico. Most of the seismic activity occurs between Albuquerque and Socorro, 65 miles north-northeast of the project. Approximately 250 earthquakes have been recorded from 1849 through 1990, often occurring in swarms. This activity is attributed primarily to the injection of magma at depth in the central part of the rift. Earthquakes of up to Modified Mercalli intensity VIII have been reported. The maximum magnitude measured was 5.1 on the Richter scale for two earthquakes which were part of a swarm in 1966 in the northern portion of the state. Accurate instrument data for magnitudes is not available before 1938, and for hypocenters before 1952. There have been no recorded earthquakes within 20 miles of the project. Earthquake data is discussed in detail in the Seismic Analysis Report completed for the original design and included in Appendix E.

2.04 Site Geology. Plate No. 6 in Appendix A is a site geologic map. Plates No. A-7 through No. A-12 in Appendix A are geologic cross sections of the project. These plates illustrate the features described in Paragraphs 2.04a, 2.04b, 2.04c, and their subparagraphs.

a. Structural Geology. The structural geology at the project site is quite complex. Through time, the area has been subjected to tensional and compressional tectonic stresses, resulting in the numerous structural features exposed at the site before and during excavation of the dam foundation trench and auxiliary spillway.

(1) Jointing. There are four primary or dominant joint sets at the site. One set (A) strikes N4°W to N10°W and dips steeply to the southwest. Another set (B) strikes N-S to N20°E and dips steeply to the northwest. A third set (C) has developed along contacts and bedding planes striking N22°W to N27°W and dipping 22° to 27° to the northeast. This set has the most joints of the four sets, but they are not as well developed. The fourth, and most dominant set (D), consists of vertical or near vertical joints striking N63°E to N69°E (photo 1). The joints in this set can be traced through the majority of the exposed column, intersecting more contacts than any of the other joints. There are two additional joint sets that are well developed but less evident. One set (E), which is more predominant on the right abutment, strikes N40°W to N50°W, dipping moderately to the southwest, and the other is a vertical to



Qal - ALLUVIUM (Holocene)--Unconsolidated sand, silt, and gravel. Includes some alluvial fan deposits.

Qtg - TERRACE GRAVELS (Pleistocene)--Four terraces have been incised into the pediment deposits. These terraces have a thin veneer of soil or sand and gravel that developed during periods of low flow.

QTp - PEDIMENT DEPOSITS (Pleistocene-Pliocene)--Unconsolidated silt, sand, and gravel, generally with well developed soil at the surface, interbedded with minor thin flow-basalt.

Tpg - PALOMAS GRAVEL (Pliocene)--Light to medium gray lenticular gravel and sandy gravel interbedded with light pink, tan, and greenish gray silt and conglomeratic sandstone, white sand, and moderate orange-red to brick red and moderate green mudstone. Several altered ash beds near top of unit, locally indurated, but generally unconsolidated.

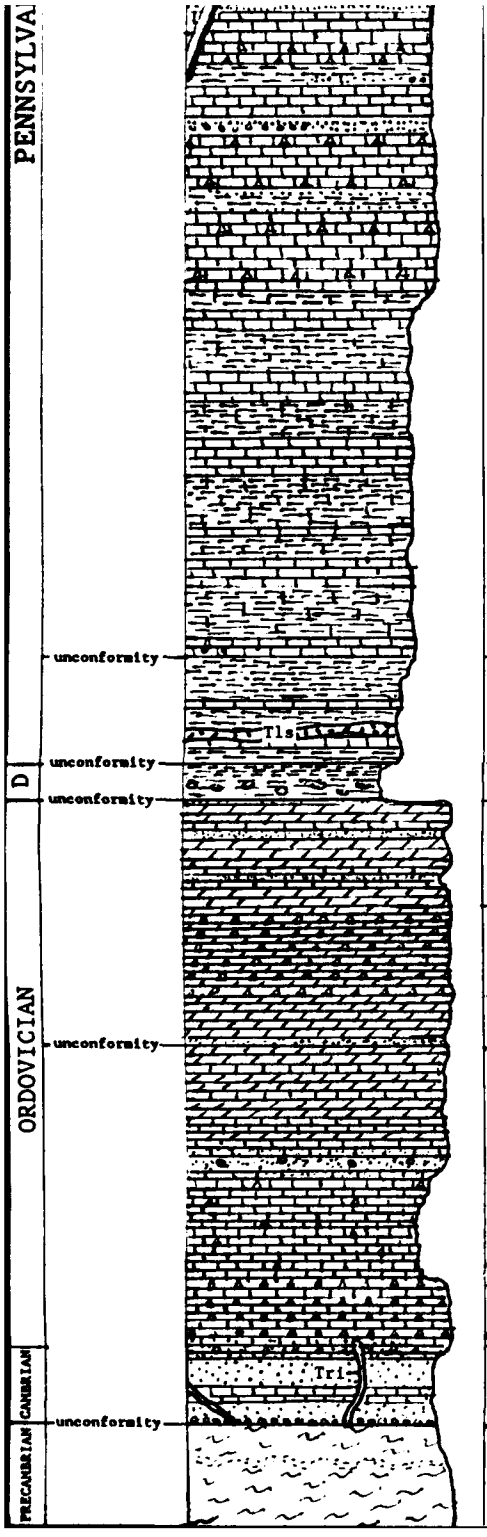
Tpf - *MUD MOUNTAIN FANGLOMERATE (Pliocene)*--Light gray fanglomerate and conglomerate composed predominantly of boulders, cobbles, and pebbles of limestone in a matrix of sandy marl; a facies of the Palomas gravel mapped separately.

Tsf - SANTA FE GROUP (Miocene)--Reddish gray, pale brown, and tan, slightly indurated conglomeratic and sandy mudstone, calcareous mudstone, and sandstone, in part tuffaceous.

Pa - ABO FORMATION (Lower Permian)--Composed predominantly of dark red shale and sandstone with minor lenses of orange-red arkosic sandstone and conglomerate and purplish gray nodular, calcareous mudstone. Formation has been intruded by Miocene basalt dikes (Tbi).

Pb - BURSOM FORMATION (Lower Permian)--Moderate red, green, and purplish gray shale and calcareous shale, minor red sandstone and gray limestone. Nodular purplish gray argillaceous limestone at base and top of unit.

PPm - MADERA FORMATION (Pennsylvanian)--Upper part: Light to dark gray, thin thin bedded limestone, locally with tan-weathering chert, alternating with dark gray, thin to thick bedded shale and minor greenish to reddish gray siltstone and shale.
Middle part: Predominantly thick to massive cherty limestone with thin dark gray shale interbeds; upper part contains local lenticular beds of dark red sandstone, conglomerate, mudstone, and dark purplish red shale.
Lower part: Gray, thin bedded but occasionally massive, locally cherty, limestone and calcarenite interbedded with light gray and green, thin to thick shale beds.
Formation has been intruded by a Miocene basalt dike (Tdi) and an Eocene porphyritic quartz latite dike (Tql).



Lower part: Gray, thin bedded but occasionally massive, locally cherty, limestone and calcarenite interbedded with light gray and green, thin to thick shale beds.
Formation has been intruded by a Miocene basalt dike (Tdi) and an Eocene porphyritic quartz latite dike (Tql).

IPPs - SANDIA FORMATION (Pennsylvanian)--Light gray and moderate greenish gray shale interbedded with light gray limestone and calcarenite and a few thin arkosic sandstone lenses.
Formation has been intruded by an Eocene latite sill (Tls).

Dp - PERCHA SHALE (Devonian)--Upper part generally dark gray fissile claystone and siltstone, locally calcareous; grades downward into light gray to buff, tan-weathering, calcareous siltstone; local thin limestone layers and nodules in lower part; local massive red-brown-weathering calcareous siltstone at base.

MONTOYA GROUP (Upper Ordovician)

Omc - CUTTER DOLOMITE--Dolomite, limestone, and siltstone; dolomite generally thick bedded, light to dark gray, and sublithographic, with nodular chert in middle and upper part; interbedded limestone is dark gray, thin bedded, microgranular; basal part is calcareous siltstone and shale. Generally unfossiliferous.

Oma - ALEMAN FORMATION--Dolomite and chert in alternating layers; medium gray, medium to coarse grained dolomite in layers 2 to 12 inches thick, weathers to brownish gray and variegated pink and white; white, brown, and black chert in irregular bands 1 to 3 inches thick, with some nodular chert in upper part.

Omu - UPHAM DOLOMITE and CABLE CANYON SANDSTONE--Upham dolomite: dark gray, weathers to brownish gray, fine grained to coarsely crystalline; minor irregular chert nodules, scattered detrital quartz grains, irregular lenses of sandstone in basal part, grades into underlying Cable Canyon sandstone: coarse grained granulitic sandstone and granule conglomerate, locally crossbedded, weathers to dark gray-brown.

EL PASO GROUP (Lower Ordovician)

Oeb - BAT CAVE FORMATION--Upper part: medium to thick bedded alternating medium gray and dark gray very fine grained limestone, dolomitic limestone, and minor dolomite; light to dark brown chert in bands and nodules; light gray stromatolitic zones; numerous solution caverns and collapse breccia. Lenticular sandstone and conglomerate locally near middle of formation.
Lower part: thin to medium bedded, light to moderate gray limestone, many stromatolitic biostromes interspersed with detrital limestone, local nodular chert, chert bands, and laminae.

Oes - SIERRITE LIMESTONE--Medium to dark gray with brownish cast, fine to very fine grained, thin to medium bedded; alternating limestone layers and thin undulating laminae of light gray to white chert resulting in wavy banded appearance.

OCb BLISS SANDSTONE (Lower Ordovician and Upper Cambrian)--Thin bedded, brown-weathering, laminated cherty limestone alternating with thin bedded glauconitic sandy shale. Most of unit alternating thin beds of gray glauconitic limestone and dolomite, and fine grained ferruginous or glauconitic sandstone and siltstone. Oolitic hematite bed 5 to 6 feet thick near base. Basal part is medium to coarse grained sandstone and conglomerate.
Formation is intruded by Eocene porphyritic rhyolite dike (Tri).

PC - PRECAMBRIAN ROCKS UNDIVIDED--Complexly contorted and interlayered reddish quartzite, dark gray and brown quartz schist, quartz-biotite schist, amphibolite gneiss and schist, and intercalated reddish porphyritic granite gneiss.

near vertical set (F) striking E-W ± 15°.Most joints can be traced over relatively long distances and are often noticeably linear. As expected, certain joint sets are more prevalent in brittle units within the bedrock and are often attenuated to the point of non-existence in the more plastic shale, mudstone, and siltstone units. The majority of joints do, however, cross at least one bedding plane contact. Only rarely are the joints of one set terminated by the joints of another set. The majority of joints show little evidence of movement as expressed by the lack of offset of intersecting joints or bedding planes. Some of the joints in the set striking N4°W to N14°W do show evidence of dip-slip movement. Offsets of one to four feet were observed. A total of 1,145 joints were mapped during foundation mapping. Figures C-1 through C-6 in Appendix C are joint rosettes and stereonets.

(2) Folding.

(i) Very gentle folding of the Paleozoic rocks can be observed at the RCC dam site. Folding developed along a N68°E trend, creating a series of minor anticlines and synclines nearly parallel to the axis of the dam (photo 2), which is situated obliquely along the crest of an anticline. Maximum relief of folding appears to be less than 10 feet, and was not indicated on the site geologic map.

(ii) At the auxiliary spillway, folding consists of a north-northeasterly trending and gently plunging syncline that has formed in the Pliocene Palomas gravel (photo 3). Given the age of deposition of the Palomas gravel, it must be assumed that folding is the result of tensional stress caused by either additional normal movement along the range-bounding fault described in Paragraph 2.03a(2), subsidence into a large linear solution cavern, or a combination of both. The nonchaotic nature of this structural feature indicates that folding occurred at a steady, slow rate during or shortly after deposition of the Palomas gravel during the Pliocene. In addition to the syncline, there is evidence of drag folding in the Paleozoic rocks exposed on the left side of the auxiliary spillway related to movement along this same fault (photo 4).

(3) Faulting. Faulting at the project site is more complex than originally described in the Design Memorandum. Faulting near the site was not noted during preliminary field studies. In addition, Pre-construction explorations were limited in number and inadvertently located so as to not intersect the faults at the site.

(i) A cross section provided with the Seismic Analysis Report in Appendix E shows a low angle thrust fault located below the axis of the dam, but no mention of it is made in the report. This fault does, however, correspond to three closely spaced thrust faults that were exposed during excavation of the left side of the auxiliary spillway. These faults are probably the oldest observed at the site. They are extinct, non-capable faults related to compressional forces that are no longer a mechanism for seismic activity in this area. The thrust faults, where exposed in the excavation, strike N35°W to N40°W and dip 35° to 45° to the northeast (photo 5). Sickensides indicate that movement along this fault was slightly rotational, having both a dip slip

and a strike slip component. In addition, the strike of joints in the hanging wall of one of the faults differs by approximately 5° from the strike of joints in the foot wall (photo 6). Assuming that the dip angle of these faults remain the same, they are approximately 500 feet below the dam at the center of the channel at an elevation of 4,110 feet (NGVD).

(ii) A number of joints in the N4°W to N14°W-striking set exhibit some minor offset. Normal dip slip movement of one foot to two feet was observed in one closely spaced group of joints in the foundation trench on the left abutment of the dam. The degree of brecciation and severe polishing observed indicate that a significantly thicker column was present during movement than what is now exposed (photo 7 & 8). Four feet of normal offset was observed approximately 1,300 feet upstream of the dam on joints within this set. An exposure on the left side of the channel downstream of the dam indicates that other joints in this set have experienced similar movement and solutioning as that observed in the foundation trench. This other group of joints, if extended, would be located approximately 60 to 80 feet into the left abutment of the dam at the dam axis. A linear feature expressed as the small valley where the maintenance/haul road was constructed also follows the strike of this set. Diamond core borings CN-CH-7 and -6 were drilled on opposite sides of this feature near the extreme left end of the earth embankment section. Depth to the top of bedrock is offset by 55 vertical feet between these two borings. If this offset is the result of normal dip-slip fault movement, the dip would have to be to the northeast, which is opposite of the dip observed in the joints of the set described in earlier in this paragraph. It is likely, therefore, that this offset is the result of the bedrock contact following bedding planes.

(iii) A group of both northeast- and northwest-striking joints between STA 3+60C and STA 4+10C exhibit evidence of movement. At least eight joints in this interval show minor (< 2 feet) normal dip-slip movement. Total displacement across this interval is approximately 10 feet. Offset decreases to the northeast and southeast. Where they cross the channel bottom downstream of the dam, two of these joints show no signs of movement and are completely healed with calcite.

(iv) Bedding Plane Fault. Another linear feature striking N80°W to N85°W is expressed as the narrow valley into which the auxiliary spillway's flows are directed. Formation dips are generally 5° steeper to the south of this feature, and logs of borings drilled along or adjacent to it indicate that it may be a fault. All of the diamond core borings drilled on this feature encountered highly fractured and severely weathered and solutioned limestone and shale. In addition, the diamond core borings drilled in 1989 for the auxiliary spillway to the north of this feature encountered a well consolidated conglomerate directly overlying the Paleozoic marine sediments. This conglomerate was not encountered in any of the borings drilled to the south of the feature.

(v) A low-angle normal bedding-plane fault was observed on the left abutment of the dam during excavation of the foundation trench. This fault is marked by a 1 foot- to 4-foot-thick sheared, brecciated, and altered shale

zone (photo 9) overlying a solutioned limestone bed. Large, joint-bounded blocks of overlying limestone and shale slid down dip along this weak zone. Total distance of movement along this fault cannot be determined, but pseudo-drag folding observed south of the dam indicates that different blocks travelled different distances, with some blocks offset across joints by as much as 75 feet. Movement also appears to be somewhat pivotal, with a hinge point located just upstream of the dam. This fault is expressed as a zone of weakness marked by increased weathering and erosion that can be traced along the entire east flank of the Mud Springs Mountains (photo 10). The amount of displacement along this fault, as well as the degree of surface expression, generally increases to the south of the project.

(vi) It appears that approximately 5,000 feet south of the auxiliary spillway, the range-bounding fault described in Paragraph 2.03a(2) turns or branches into another fault or zone of en echelon faults that strike N13°E. This fault zone is traceable through the auxiliary spillway and appears as a dominant linear feature northeast of the project site. Maxwell and Oakman (1986) mapped this feature as a possible or inferred fault extending northeast of the project. This fault was not observed during the initial seismic study or preliminary field studies. The offset along this fault, as well as the depth to bedrock on the hanging wall side in the reservoir area are not known. At the auxiliary spillway, bedrock on the hanging wall side is as much as 90 feet to 100 feet below the crest elevation. This fault has possibly been subjected to two or more periods of activity. Brecciation, manganese mineralization, and partial healing of this fault was observed upstream of the dam on the right side of the channel, and jasperoid-healed breccia was observed along this fault in a small canyon southwest of the auxiliary spillway. This brecciation and subsequent mineral deposition is related to one period of activity of unknown age. Deformation of the Palomas gravel as discussed in Paragraph 2.04a(2)(ii), as well as primary calcite mineralization observed in the foundation of the auxiliary spillway, indicate that another segment of this fault zone experienced a separate period of movement and hydrothermal activity as a result of continued rift activity during the Pliocene.

b. Stratigraphy. Stratigraphy at the site, itself, is relatively simple to interpret and describe. Correlation of rock units between points on the project is excellent, with only minor thinning and/or pinching out of various units within the exposed formations. The marine sedimentary rocks of the Pennsylvanian Madera formation are the only pre-Tertiary rocks exposed at the project. For this report, the Tertiary Santa Fe group and Palomas gravel are included as bedrock units due to the high degree of consolidation of some beds within these units, as well as the fact that the Palomas gravel forms the foundation for the earth embankment section of the dam and the majority of the auxiliary spillway. Overburden at or near the project consists of Pliocene pediment deposits, Pleistocene terrace deposits, and Holocene alluvium and slopewash. Table C-1 in Appendix C contains detailed lithologic descriptions of the Madera formation rock units exposed during excavation of the foundation trench for the RCC dam and the left side and left abutment of the auxiliary spillway. Table C-2 in Appendix C contains detailed lithologic descriptions of the Santa Fe group and Palomas gravel exposed during excavation for the

majority of the auxiliary spillway and the inspection trench for the embankment dam. The mapped units described in these tables are numbered to correspond with the foundation maps in Appendix D. Alluvium in the channel bottom and minor slopewash were the only overburden materials encountered during excavation.

(1) Bedrock.

(i) MADERA FORMATION (Pennsylvanian): By far, the largest exposure of bedrock at the site is the Pennsylvanian Madera formation. Excavation of the RCC dam foundation and the left side of the auxiliary spillway has exposed a column thickness of approximately 270 feet with an additional thickness of 45 feet between these two features for a total thickness of 315 feet. Because of thrust faulting and low angle bedding plane faulting, the true thickness of the Madera formation at the project is not known. Maxwell and Oakman (1986) report a thickness of approximately 1,500 feet for this area. The rocks exposed at the project represent the upper part of the Madera formation and consist primarily of gray to reddish gray, medium-bedded, micritic to argillaceous (shaley) limestone with minor dark gray to brown, thin- to medium-bedded, calcareous shale and claystone and greenish gray and reddish gray, non- to slightly calcareous siltstone (photos 11, 12, 13, & 14). The shale and claystone beds, though well consolidated, slake readily when exposed to the air (photos 15 & 16). One thin bed of sandstone and some minor chert were also exposed on the left abutment of the dam. In recent geologic reports and publications, the Madera formation in southern New Mexico has been reclassified as the Bar B formation and the Nakaye formation.

(ii) SANTA FE GROUP (Miocene): The Santa Fe group is not exposed at the site, but a well consolidated conglomerate member of this formation was encountered in borings drilled at the auxiliary spillway. It is exposed in only one outcrop on the right side of the Cuchillo Negro Creek channel one mile upstream of the dam. It consists primarily of slightly consolidated, reddish gray to pale brown conglomerate, sandstone, mudstone, and conglomeritic and sandy mudstone with minor tuffaceous ash near the top of the section. At this outcrop, it has been tilted and faulted, dipping 30° to 50° to the southwest.

(iii) MUD MOUNTAIN FANGLOMERATE (Pliocene): A fanglomerate deposited directly onto the Madera formation limestone bedrock along the western flank of the Mud Springs Mountains was exposed at the left abutment of the auxiliary spillway and the left abutment of the RCC dam during excavation (photo 17). It is slightly to well consolidated, light gray to tan, and composed of subangular limestone boulders, cobbles, and pebbles in a sandy marl matrix. Well consolidated fanglomerate was left in place as suitable foundation material.

(iv) PALOMAS GRAVEL (Pliocene): The earth embankment and most of the auxiliary spillway are founded on the Palomas gravel. It is composed of light to medium gray conglomerate and sandy conglomerate interbedded with moderate brown to reddish brown mudstone and minor gray sandstone. Individual units within this formation vary from unconsolidated to well consolidated, primarily

as a function of depth (photo 18). There are a few ash beds near the top of the formation. It consists of flood plain and playa deposits that grade laterally into river channel and coarse fan deposits. It overlies the Santa Fe group and the Mud Mountain fanglomerate with angular unconformity. Although generally flat lying, it has been deformed into a synclinal structure at the auxiliary spillway (photo 3).

(2) Overburden.

(i) TERRACE GRAVELS (Pleistocene): Maxwell and Oakman (1986) mapped four terraces of Pleistocene age along the sides of the Cuchillo Creek valley. None of this material was identified during excavation of the foundation trench or the earth embankment inspection trench. These deposits are, however, exposed nearby both upstream and downstream of the project. Most of these terraces have a thin veneer of soil or sand and gravel that developed during periods of low flow. Each preceding terrace was then incised during periods of high flow associated with glacial stages.

(ii) ALLUVIUM (Holocene): Unconsolidated sand, silt, gravel, and minor cobbles and boulders have been deposited in the bottom of Cuchillo Negro Creek and tributaries. Gravel clast is predominantly composed of generally hard, somewhat tabular, subrounded, igneous rocks with imbricate deposition. The aggregate for all concrete for the project was from this material. It is 30 feet to 40 feet thick at the damsite, and of undetermined thickness in the wide valley borrow area upstream of the dam.

(iii) SLOPEWASH (Holocene): A thin veneer of colluvium, 1 inch to 1 foot thick, overlies much of the slopes formed on the limestone bedrock where it is not covered by the Palomas gravel. It is composed of unconsolidated angular limestone cobbles and gravel with minor sand.

c. Alteration and Solutioning. Alteration of the limestone varied from slight to severe. All alteration and subsequent solutioning developed along or adjacent to joints and bedding planes.

(1) Alteration. Alteration was most severe in limestone directly underlying shale beds. Low pH hydrothermal fluids moved along joints and some bedding planes, altering the limestone to a slightly cemented calcareous silt and/or dissolving calcareous siltstone beds and previously altered limestone. These fluids were trapped by the impervious shale, resulting in the development of large alteration zones in the limestone directly below these beds, especially along joints which had previously experienced movement and subsequent brecciation of adjacent rock. Three zones of alteration were exposed during excavation for the project: one on the left abutment of the RCC dam, one on the right abutment of the RCC dam, and the third on the left side of the auxiliary spillway. At the dam, severe alteration developed adjacent to joints in the limestone underlying a brecciated shale shear zone, and was most severe along a closely spaced group of N8°W to N12°W trending joints that dip 60°SW to 65°SW which were exposed on the left abutment (photo 19). Only the basal portion of the zone of alteration was exposed on the right abutment, as all overlying strata was previously eroded. At the auxiliary spillway,

severe alteration developed along all joints in the limestone underlying a thick shale unit (photo 20) near where the thrust fault is splintering. With depth, the degree of alteration of the limestone in both zones decreased significantly. Eventually, the hydrothermal fluids cooled, became saturated, and began to sink, depositing calcite and healing most open joints.

(2) Solutioning. On the left abutment of the dam and the left side of the auxiliary spillway, solutioning developed in the areas of severe alteration. A large solution cavity was encountered during excavation of the foundation trench on the left abutment of the RCC dam. Solutioning developed along the joints described in Paragraph 2.04a(3)(ii) and in the zone of alteration described in Paragraph 2.04c(1). This large cavity was nearly 30 feet wide at the top, narrowing to less than 6 inches at a depth of 25 feet, and extended through the excavated width of the foundation trench. It was filled with scattered fragments of unaltered limestone and shale material which had collapsed from the ceiling and clay washed into the cavity by groundwater (photo 21). Very few open areas remained.

(i) On the left side of the auxiliary spillway, a network of solution cavities developed along the joints of the limestone in the alteration zone described in Paragraph 2.04c(1). The solution cavities were all subsequently filled with sandy clay deposited by groundwater.

(ii) On the right abutment of the dam, the alteration of some of the limestone was just as severe as that of similar units exposed on the left abutment. Being higher in elevation, however, the alteration zone on the right abutment was not subjected to solutioning by groundwater. As a result, the majority of the altered limestone adjacent to joints remained in place (photo 22). Bedding planes and other features can be traced through the altered areas (photo 23). This material was removed from the wider zones during excavation and replaced with dental concrete.

(iii) Small scale features were often seen to control the development of other alteration and solutioning. Severe alteration and solutioning were often found to terminate at joint intersections and joint/bedding plane intersections (photo 24). Occasionally, however, solutioning would continue through bedding and/or across joints. There were some instances where alteration and subsequent solutioning terminated along a joint for no apparent reason (photo 25). Some rock beds were more resistant to alteration than others. Well-developed solution cavities formed along joints above and below these resistant beds, but the beds themselves were not subjected to alteration and subsequent solutioning. These factors often made it difficult to determine at what point removal of unsatisfactory material could be terminated.

d. Seismicity. A seismic analysis was conducted in 1986 by Tierra Engineering Consultants, Inc. The results of this analysis were based on field observations and data from Maxwell and Oakman (1986), as well as others. The Seismic Analysis Report is included in Appendix E. A report on the probability of the combined occurrence of a seismic event and flood risk-storage is also contained in Appendix E. This report was originally included

as Section 5 of the General Design Memorandum. The analysis judged that range-bounding, northwest-trending faults of the Mud Springs Mountains were capable based on the relationship between these faults and present tectonic rift activity. This criteria was used for the selection of the maximum earthquake. A northwest-trending fault located approximately 6,500 feet upstream of the dam was selected as the fault that ruptures. This fault is not illustrated on the Geologic Map of the Cuchillo Quadrangle by Maxwell and Oakman (1986). They have shown the range-bounding fault to be a somewhat discontinuous set of en echelon faults that cross the channel approximately 5 miles upstream of the project. It was determined, in the seismic analysis, that the maximum earthquake that could occur on the capable fault would be felt at the Cuchillo Negro Dam site as an Intensity VIII or magnitude 6 event. This event could produce peak accelerations of up to 0.4 G's, a peak ground velocity of up to 1.3 ft/sec, and a peak displacement of up to 0.5 feet, with a duration of 11 seconds.

Maxwell and Oakman (1986) have mapped a number of north-northeast striking faults that branch off the set of northwest striking faults described above. One of these branching faults forms the escarpment of the northern end of the Mud Springs Mountains and can be extended through the project. This is the fault described in Paragraph 2.04a(3)(vi), which appears to have experienced movement as the result of continued rift activity. This fault, therefore, may be a capable fault located substantially closer to the project than the capable fault assumed in the seismic analysis. The field evidence observed to date indicates that the last activity along this fault occurred during or shortly after the Pliocene, 3 to 4 million years ago. Given the lack of significant post-Pliocene deposition, it may not be possible to accurately date this fault.

e. Groundwater. The overall groundwater gradient in the region is toward the Rio Grande River to the east with a somewhat southerly component paralleling the Rio Grande Valley. Depth to groundwater in the project area is extremely variable, being directly correlatable to seasonal changes and the amounts of precipitation and run off. Depth to groundwater is generally lower during early summer than in late winter. During the early spring of 1989, groundwater in the channel bottom alluvium was encountered at depths of five feet to eight feet. In March, 1990, after a year of less-than-normal precipitation, the Contractor drilled a water supply well in the valley bottom approximately 1,500 feet upstream of the dam. Groundwater was first encountered at a depth of 55 feet. The driller estimated a pump rate of three to four gallons per minute. This well was drilled to the limestone bedrock at a depth of 145 feet and continued to a final depth of 340 feet without encountering any significant amounts of additional water. The well was abandoned when it was determined that it could not supply the Contractor with the quantities of water needed for construction. The driller's log is included in Appendix B.

(1) During excavation of the foundation trench, a number of relatively small floods came down the valley, flooding the excavation and completely filling it with alluvial material (photos 26 & 27). This material remained saturated and was difficult to excavate due to its quicksand-like properties

(photo 28). Groundwater flowed through this material following the surface topography. When levels of groundwater dropped to depths below the top of bedrock, groundwater could no longer be seen, and the bottom of the foundation trench remained dry. Any water which ponded on the bedrock foundation after a rainstorm would remain until it was pumped out or evaporated. It appears that when groundwater levels drop to below the top of bedrock, the buried fault scarp of the Mud Springs Mountains acts as an impervious barrier to horizontal movement of groundwater toward the east. At depth, the groundwater is diverted to the south, flowing parallel to the fault scarp until it reaches the Palomas Basin.

(2) Because of the tightness of the bedrock and the diversion of groundwater to the south, the uplift pressures on the structure during impoundment are likely to be less than those anticipated during design.

2.05 Engineering Characteristics of the Overburden and Bedrock Materials. The engineering characteristics of overburden and bedrock materials, including results of tests, are discussed in Section III, Investigations.

2.06 Unusual or Unanticipated Geologic Conditions Encountered During Construction. There were several unusual and unanticipated geologic conditions encountered during construction which resulted in modifications to the contract and differing site condition claims. These conditions were encountered during excavation of the auxiliary spillway and the RCC dam foundation trench.

a. RCC Dam Section. The most significant impact to the contract was caused by the unanticipated geologic conditions that were exposed during the excavation of the RCC dam foundation trench. Expected geologic conditions were based on field observations and interpretation of the drill logs of a limited number of preconstruction exploration borings. No angle borings, which may have provided additional information regarding steeply dipping joints, were drilled at the site. Other than one boring located at the intake tower, and one boring located in the stilling basin, no borings were drilled to explore the foundations of the appurtenant structures. Preconstruction investigations are described in detail in Section III, Investigations. Plate No. A-14 in Appendix A is a site plan map that illustrates the locations of explorations. Plate No. A-15 in Appendix A is a cross section along the dam axis that compares the as-designed excavation with the as-built excavation.

(1) Left Abutment. Only one boring, CN-CH-1, was drilled on the left abutment of the RCC dam section and is located 65 feet downstream of the dam axis. The location of CN-CH-1 and an error in spotting the boring on investigation plans allowed 110 feet of the stratigraphic column on the left abutment to remain unexplored when construction started. This portion of the stratigraphic column was exposed in the left abutment between the high level outlet works and the bottom of the dam. The bedding plane fault and associated shear/breccia zone described in Paragraph 2.04a(3)(v), the joints with evidence of movement and associated brecciation described in Paragraph 2.04a(3)(ii), and the alteration and solution zone described in Paragraph 2.04c and subparagraphs were all located in a 70-foot-thick section of this

interval (photo 29). This interval also contained several shale beds which slaked readily when exposed to the air (photos 15 & 16). Most joints were steeply dipping and were not, therefore, encountered in CN-CH-1. As a result, a significant amount of the material encountered during excavation of the foundation trench on the left abutment of the RCC dam was of poorer than expected quality. Figure 2-2 is a cross section which illustrates this interval. The contractor filed a differing site conditions claim, and modification Nos. P00025, P00037, P00045, and P00055, totaling $1,342,238.00 were issued to alter the excavated slope on the left abutment. Details of this excavation are discussed in Section V, Excavation. Modification Nos. P00009 and P00019 for $3,984.00 were issued to provide for additional foundation explorations. Modification Nos. P00031 and P00034 for $229,661.00 were issued to install control joints in the RCC dam. Also, over 1,500 cubic yards of dental concrete was required to fill cavities and irregularities in the foundation surface for the left abutment of the RCC dam (photo 30).

(i) High Level Outlet Works.

(*a*) A siltstone bed, 3 to 5 feet thick, with thin shale partings was encountered during excavation of the foundation for the box culvert section of the high level outlet works (HLOW). This material was sheared and slaked readily when exposed to the air (photo 12). It was determined to be of insufficient quality for use as a foundation and was removed. Approximately 500 cubic yards of dental concrete was required to reconstruct the foundation to invert grade (photo 31).

(*b*) The elevations for the as-designed cut slopes on the left abutment fell nearer to the original ground surface than what was illustrated on the contract drawings. In addition, as discussed in Paragraph 2.04a(1), the bedrock was highly jointed. As a result, the bedrock, which was to form the right side of the HLOW discharge channel and the foundation for the HLOW intake and trash rack, was of insufficient width and/or quality to support these structures (photo 32). Work was suspended on the HLOW and modification Nos. P00015, P00018, and P00023 were issued to construct a concrete "U" shaped discharge channel and to redesign and relocate the trash rack (photo 33). These modifications, suspensions, and subsequent claims added $95,085.00 to the cost of the contract.

(ii) Low Level Outlet Works. A similar situation was encountered during excavation of the foundation for the conduit section of the low level outlet works (LLOW) (photo 34). The bedrock was originally intended to be the forming for encasement of the conduit but had to be removed due to insufficient width and quality. Approximately 300 cubic yards of dental concrete was required to reconstruct the foundation, and modification No. P00035 was issued to provide for the use of forming for the concrete encasement of the LLOW conduit (photo 35). This modification added $22,500.00 to the cost of the contract.

(2) Right Abutment.

(i) The bedding plane fault and associated shear/breccia zone discussed

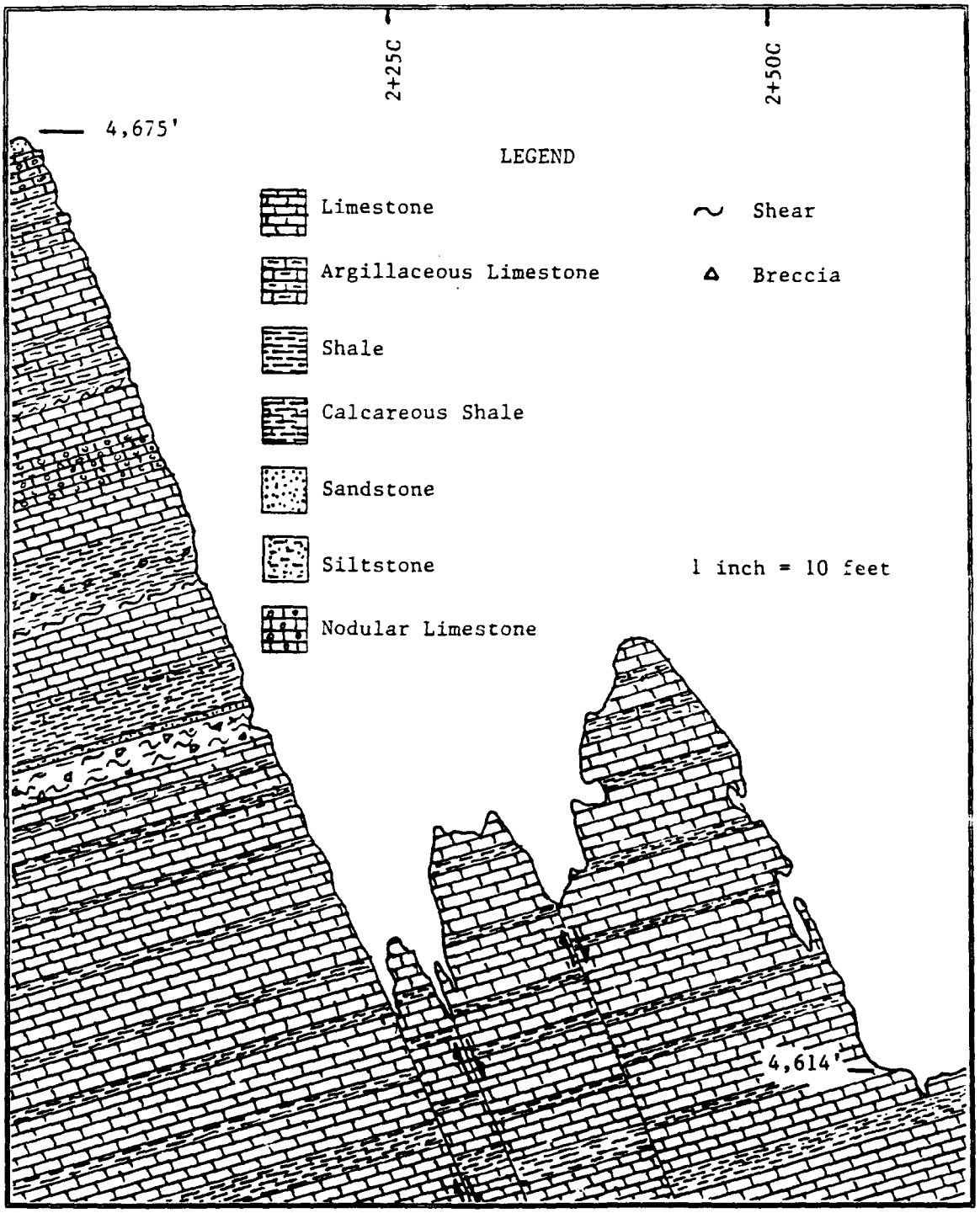


Figure 2-2 Cross Section of Left Abutment Between the HLOW and the LLOW

in Paragraph 2.04a(3)(v) and the highly jointed bedrock and shale beds discussed in Paragraph 2.04a(1) have been eroded from the slope forming the right abutment of the dam. The base of the altered limestone zone described in Paragraph 2.04c(1) was, however, exposed during excavation of the foundation trench on the right abutment. The altered limestone and joint infilling material was removed and replaced with dental concrete (photo 36). Approximately 500 cubic yards of dental concrete were required in this area.

(ii) The attitude of the joints encountered during excavation, and the tendency for the bedrock to break along bedding planes resulted in a "stairstep" type foundation surface for much of the floor of the foundation trench on the right abutment with numerous, although relatively small, "negative" slopes. A significant amount of dental concrete was required to reshape these slopes in order to accommodate RCC placement. This same situation was encountered on the left abutment, but the result, in this case, was the formation of numerous trough-like features that were easily filled with dental concrete. Figure 2-3 is a schematic diagram of these left and right abutment features.

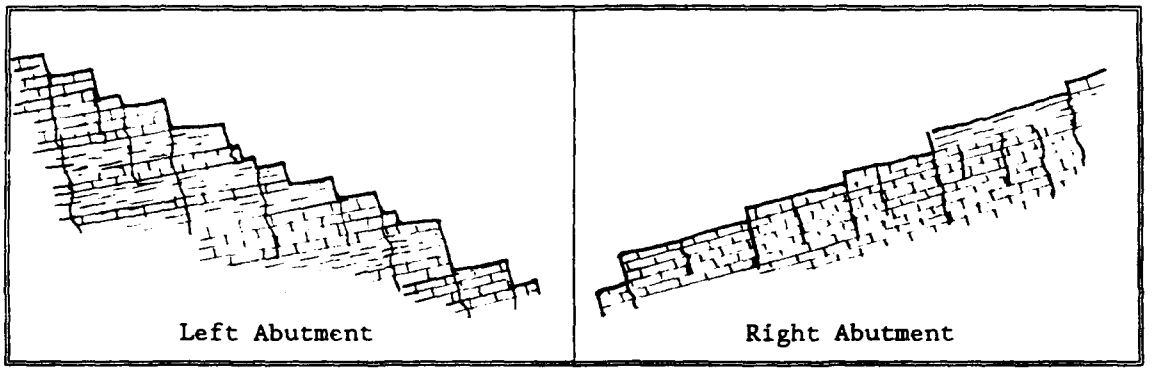


Figure 2-3 Schematic Diagram of "Stair Steps" on the Floor of the RCC Dam Foundation Trench

b. Auxiliary Spillway. The Palomas gravel forms the majority of the foundation for the auxiliary spillway. Regionally, it is a relatively horizontal layer separated from the older underlying sediments by an angular unconformity. During excavation for the spillway, it was exposed as a syncline plunging gently to the northeast. As discussed in Paragraph 2.04a(2)(ii), the syncline appears to have formed contemporaneously with deposition of the Palomas gravel or shortly thereafter due to the lack of any shear or other chaotic features in even the well consolidated beds within the formation.

(1) During excavation for the auxiliary spillway in January 1990, the Contractor encountered significant amounts of a well consolidated conglomerate bed within the Palomas gravel (photo 37). This material could not be ripped using the equipment specified, and the Contractor claimed that it would have to be classified as rock excavation (photo 38). The specifications for the contract stated that the chute toe of the auxiliary spillway may be founded on

the consolidated conglomerate beds encountered during preconstruction explorations. A determination was made to found the chute toe on this material at an elevation of 4,687 feet (NGVD). The as-designed chute toe elevation was 4,628 feet (NGVD). Although a significant reduction in the quantities of common excavation, random fill, and RCC placement was realized as a result of the new chute toe elevation, the Contractor filed claims related to delays in excavation and having to rehandle material that had been wasted, as well as variation in estimated quantities. The Contractor claimed that there was not enough fill material stockpiled for use in the earth embankment section, and previously wasted material had to be retrieved. Modification No. P00056 was issued to settle the change-in-design claim which added $80,000.00 to the cost of the contract.

(2) A large area of solutioned limestone and poor quality rock was encountered during rock excavation on the left side of the auxiliary spillway. This solutioned zone is discussed in Paragraph 2.04c(2)(i) (photo 39). Over 1,000 cubic yards of dental concrete were required to reshape the foundation in this area (photos 40 & 41).

(3) Table 2-1 is a list of contract modifications that were a direct or indirect result of foundation conditions.

Table 2-1 Contract Foundation Modifications

Mod Number	Description	Cost ($)
P00000	Variations in Estimated Quantities*	763,363
P00009	Exploration Drilling on the Left Abutment	2,846
P00015	Construct Concrete Channel for HLOW	57,756
P00018	Revise and Construct Trash Rack/Entrance to HLOW	28,300
P00019	Replace Drill Steel	1,138
P00023	Finalization of P00007 and P00010, Suspend Work	9,029
P00025	Finalize P00022, Benching/Material Removal	684,442
P00031	RCC Dam Control Joints	85,531
P00034	Extension of Control Joints	144,130
P00035	Framing of LLOW Conduit Encasement	22,500
P00037	Unilateral Adjustment for P00022	95,000
P00043	Rock Bolts, Right Abutment (Claim)	5,400
P00045	Increase to P00025	175,000
P00055	Remove Differing Site Conditions, Left Abutment	387,796
P00056	Spillway Claim	80,000
	Total Cost of Contract Foundation Modifications:	$2,542,231

* The Majority of Variations in Estimated Quantities Were the Result of Unanticipated Geologic Conditions

III - FOUNDATION INVESTIGATIONS

3.01 Investigations Prior to Construction. Subsurface investigations were conducted at the project site by the Albuquerque District in 1984, 1986, 1987, 1988, and 1989. Explorations consisted of diamond core borings, 8-inch-diameter auger borings, air rotary borings, down hole air hammer borings, a Denison boring, and backhoe trenches. In addition to the subsurface explorations, surface seismic refraction tests were conducted by Waterways Experiment Station (WES) in 1986, a borehole photography study was conducted by Walla Walla District in 1987 and by Southwestern Division Laboratory in 1988 and 1989, and downhole geophysical logging was conducted by the U.S. Geological Survey in 1988. Laboratory tests were performed on soil and rock core samples collected from the borings. Table 3-1 provides pertinent data regarding the preconstruction subsurface explorations. Detailed boring logs of the subsurface explorations are contained in Appendix B. The logs for CN-T-19, CN-T-20, CN-CH-29 through -33, and R-1 through R-3 were not located. The downhole geophysical summary report and logs, laboratory test results, and the borehole photography reports are contained in Appendix E. Plate No. A-14 in Appendix A illustrates the location of all the subsurface explorations except those which fell outside the map boundaries.

a. Subsurface Explorations.

(1) Explorations in 1984 were conducted in April and May and consisted of four diamond core borings and nine backhoe trenches. HQ (2.5-inch-diameter) diamond core borings CN-CH-1, -2, and -3 were drilled along the proposed RCC dam alignment, and CN-CH-4 was drilled on the left abutment of the 800-foot-wide approach channel for the original spillway. The nine backhoe trenches, CN-T-5 through -13, were excavated within the limits of the flood plain, upstream of the dam site. Trenches CH-T-1 through -4 were not excavated because right of entry was not obtained.

(2) Explorations in 1986 were conducted in May, June, November, and December and consisted of ten NX (2.155-inch-diameter) diamond core borings and two 4-inch-diameter mud rotary borings. Diamond core borings CN-CH-5, -5A, -6, -7, -7A, and -8 were drilled in the high ground north and east of the dam site. CN-CH-5A and -7A were drilled approximately 10 feet from CN-CH-5 and -7, respectively, in order to obtain core samples to verify the results of the seismic survey performed by WES earlier in the year to determine depth to the top of rock on the left abutment of the dam. CN-CH-9 through -12 were drilled in the area of the originally proposed spillway. CN-CH-13 and -14, the 4-inch mud rotary borings, were drilled in the area of the 800-foot-wide spillway approach channel.

(3) Explorations in 1987 were conducted in March and consisted of seven backhoe trenches. Trenches CN-T-14 through -20 were excavated in the proposed 800-foot-wide approach channel cut between the main reservoir and the spillway. These trenches were excavated to determine the suitability of the materials from required excavation for use in the RCC and embankment sections.

Table 3-1 Preconstruction Subsurface Explorations Pertinent Data

Number	Type	Depth in Feet	Location			Feature Explored & Year Complete	
			East	North	elev. (feet)		
CN-CH-1	HQ Diamond Core	100.5	807,555	658,405	4716	1	1984
CN-CH-2	HQ Diamond Core	62.0	807,474	658,226	4618	1	1984
CN-CH-3	HQ Diamond Core	79.0	807,380	658,017	4741	1	1984
CN-CH-4	HQ Diamond Core	72.0	807,225	657,555	4732	2	1984
CN-T-5	Backhoe Trench	8.0	808,715	656,525	4650	8	1984
CN-T-6	Backhoe Trench	9.0	808,430	656,410	4640	8	1984
CN-T-7	Backhoe Trench	9.5	808,175	656,285	4648	8	1984
CN-T-8	Backhoe Trench	13.5	808,530	656,940	4648	8	1984
CN-T-9	Backhoe Trench	9.1	808,290	656,785	4638	8	1984
CN-T-10	Backhoe Trench	11.0	808,000	656,660	4642	8	1984
CN-T-11	Backhoe Trench	9.5	808,360	657,305	4639	8	1984
CN-T-12	Backhoe Trench	6.1	808,120	657,230	4633	8	1984
CN-T-13	Backhoe Trench	9.1	807,850	657,350	4632	8	1984
CN-CH-5	NX Diamond Core	50.0	807,835	658,610	4721	3	1986
CN-CH-5a	NX Diamond Core	62.5	807,835	658,610	4721	3	1986
CN-CH-6	NX Diamond Core	107.5	808,160	658,930	4743	3	1986
CN-CH-7	NX Diamond Core	50.0	808,195	658,820	4741	3	1986
CN-CH-7a	NX Diamond Core	47.0	808,195	658,820	4741	3	1986
CN-CH-8	NX Diamond Core	98.0	808,510	658,505	4719	4	1986
CN-CH-9	NX Diamond Core	126.0	807,045	657,880	4721	5	1986
CN-CH-10	NX Diamond Core	99.8	806,690	657,790	4665	5	1986
CN-CH-11	NX Diamond Core	126.0	806,375	657,830	4706	5	1986
CN-CH-12	NX Diamond Core	100.4	806,680	657,895	4662	5	1986
CN-CH-13	4-inch Rotary	50.0	807,218	657,404	4745	6	1986
CN-CH-14	4-inch Rotary	50.0	807,095	657,060	4749	6	1986
CN-T-14	Backhoe Trench	6.0	807,210	657,355	4750	6	1987
CN-T-15	Backhoe Trench	4.0	807,130	657,100	4730	6	1987
CN-T-16	Backhoe Trench	5.0	807,140	657,180	4740	6	1987
CN-T-17	Backhoe Trench	3.0	806,990	656,955	4720	6	1987
CN-T-18	Backhoe Trench	3.0	807,120	657,210	4745	6	1987
CN-T-19	Backhoe Trench	2.0	?	?	?	6	1987
CN-T-20	Backhoe Trench	2.5	?	?	?	6	1987
CN-CH-15	HQ Diamond Core	81.2	807,275	658,352	4615	7	1988
CN-CH-16	HQ Diamond Core	120.4	807,538	658,198	4617	1	1988
CN-CH-17	HQ Diamond Core	69.5	807,623	658,535	4722	3	1988
CN-CH-18	HQ Diamond Core	119.8	807,430	657,960	4740	1	1988
CN-CH-19	HQ Diamond Core	100.5	806,625	658,165	4659	5	1988
CN-CH-20	HQ Diamond Core	96.9	806,585	658,090	4660	5	1988
CN-CH-21	HQ Diamond Core	137.4	806,290	658,255	4710	5	1988
CN-CH-22	HQ Diamond Core	101.0	806,086	658,246	4756	5	1988
CN-CH-23	HQ Diamond Core	100.5	806,782	658,120	4662	5	1988

Table 3-1 cont'd

CN-CH-24	HQ Diamond Core	43.2	807,235	657,598	4730	2	1988
CN-CH-25	HQ Diamond Core	81.2	806,645	658,270	4655	5	1988
CN-CH-26	HQ Diamond Core	98.1	?	?	?	5	1988
CN-CH-27	HQ Diamond Core	50.4	?	?	?	5	1988
CN-8A-01	8-inch Auger	35.0	807,110	657,055	4748	6	1988
CN-8A-02	8-inch Auger	34.0	807,130	657,210	4746	6	1988
CN-8A-03	8-inch Auger	39.0	807,120	657,325	4739	6	1988
CN-8A-04	8-inch Auger	29.0	807,050	657,320	4732	6	1988
CN-8A-05	8-inch Auger	24.0	807,130	657,545	4724	6	1988
CN-8A-06	8-inch Auger	14.0	806,530	657,350	4677	6	1988
CN-8A-07	8-inch Auger	8.0	806,770	657,565	4671	6	1988
CN-8A-08	8-inch Auger	8.0	806,665	658,235	4656	5	1988
CN-8A-09	8-inch Auger	8.0	808,070	657,600	4628	8	1988
CN-8A-10	8-inch Auger	14.0	807,600	658,205	4618	8	1988
CN-8A-11	8-inch Auger	14.0	807,170	658,450	4613	9	1988
CN-8A-12	8-inch Auger	14.0	806,695	658,660	4596	9	1988
CN-8A-13	8-inch Auger	14.0	806,690	659,300	4601	9	1988
CN-8A-14	8-inch Auger	14.0	808,090	657,200	4633	8	1988
CN-8A-15	8-inch Auger	14.0	808,145	656,860	4639	8	1988
CN-8A-16	8-inch Auger	14.0	808,575	656,525	4643	8	1988
CN-CH-26	HQ Diamond Core	98.1	?	?	?	5	1988
CN-CH-27	HQ Diamond Core	50.4	?	?	?	5	1988
CN-CH-28	HQ Diamond Core	76.0	807,544	658,212	4620	10	1989
CN-CH-29	HQ Diamond Core	145.7	806,628	658,051	4660	5	1989
CN-CH-30	4" Air Hammer	181.7	?	?	?	1	1989
CN-CH-31	4" Air Hammer	181.7	?	?	?	1	1989
CN-CH-32	4" Air Hammer	?	?	?	?	6	1989
CN-CH-33	4" Air Hammer	?	?	?	?	6	1989
CN-CH-34	NX Diamond Core	29.5	806,762	657,701	4668	6	1989
CN-CH-35	NX Diamond Core	32.0	806,796	657,450	4675	6	1989
CN-CH-36	NX Diamond Core	28.0	806,687	657,270	4686	6	1989
CN-CH-37	NX Diamond Core	48.0	?	?	?	6	1989
CN-CH-38	NX Diamond Core	93.1	?	?	?	6	1989
CN-CH-39	NX Diamond Core	42.7	806,969	657,641	4708	6	1989
CN-CH-40	NX Diamond Core	85.7	806,843	657,123	4728	6	1989
CN-CH-41	NX Diamond Core	50.0	807,194	657,518	4740	6	1989
CN-CH-42	NX Diamond Core	97.7	806,860	657,236	4729	6	1989
CN-CH-43	NX Diamond Core	139.5	807,120	657,170	4742	6	1989
CN-CH-44	NX Diamond Core	70.0	807,086	657,548	4716	6	1989
CN-CH-45	NX Diamond Core	110.0	807,050	657,444	4723	6	1989
CN-D-1	6-inch Denison	94.8	?	?	?	5	1989
R-1	4" Air Rotary	18.1	?	?	?	8	1989
R-2	4" Air Rotary	22.8	?	?	?	8	1989
R-3	4" Air Rotary	24.1	?	?	?	8	1989

Note 1. Locations and elevations scaled from 100 Scale Project Layout Plan

Table 3-1 cont'd

Note 2. Feature Explored Symbols are as follows:

1. RCC Dam Section	6. Auxiliary Spillway
2. Auxiliary Spillway, Left Abutment	7. Dam Stilling Basin
3. Earth Embankment Section	8. Upstream Channel
4. Left Abutment, Upstream	9. Downstream Channel
5. Original Spillway	10. Low Level Outlet Intake Tower

Note 3. Missing information is denoted with a question mark (?).

(4) Explorations in 1988 were conducted in February through May and in December and consisted of sixteen 8-inch-diameter auger borings and thirteen NX diamond core borings. Auger borings, CN-8A-1 through -5 were drilled in the original spillway approach channel cut; CN-8A-6 and -7 were drilled in the area between the original spillway approach channel and the original spillway location; CN-8A-8 was drilled downstream of the original spillway location; and CN-8A-11, -12, and -13 were drilled in the channel bottom downstream of the dam site. The remaining auger borings, CN-8A-9, -10, -14, -15, and -16 were drilled in the flood plain upstream of the dam site. Diamond core borings CN-CH-16, -17, and 18 were drilled along a proposed alternate alignment of the RCC dam; CN-CH-19, -20, -23, and -25 were drilled in the valley area of the original spillway; CN-CH-21 and -22 were drilled at the right abutment of the original spillway; CH-CN-24 was drilled on the left abutment of the original spillway approach channel to the original spillway; and CN-CH-15 was drilled at the proposed outlet works stilling basin site. In December, two additional diamond core borings, CN-CH-26 and -27 were drilled in the vicinity of the original spillway.

(5) Explorations in 1989 were conducted in January, February, and April and consisted of fourteen NX diamond core borings, four 4-inch air hammer borings, three 4-inch air rotary borings, and one 6-inch Denison boring. Diamond core boring CN-CH-28 was drilled at the location of the intake tower for the low level outlet works; CN-CH-29 was drilled in the vicinity of the spillway; and CH-CN-34 through -45 were drilled at the relocated spillway in the former approach channel to the original spillway. The Denison boring, CN-D-1, was drilled at the original spillway site. Three air rotary borings, R-1, R-3, and R-4, were drilled in the channel bottom upstream of the dam, primarily to determine overburden thickness. Two downhole air hammer borings, CN-CH-30 and -31, were drilled at the RCC dam site, and two, CN-CH-32 and -33 were drilled at the original spillway site for the borehole camera study.

b. Geophysical and Borehole Photography Investigations.

(1) Investigations in 1986 were conducted during the period 25 March through 1 April. Personnel from the Earthquake Engineering and Geophysics Division, Geotechnical Laboratory, U.S. Army Engineers Waterways Experiment Station (WES), conducted surface seismic refraction tests to determine the

compression-wave (P-wave) velocities of the in-situ material and the depth to competent rock in the left abutment of two proposed dam alignments.

(2) Investigations in 1987 were conducted in September and consisted of a borehole photography study. Walla Walla District, U.S. Army Corps of Engineers performed the study to determine the orientations and apertures of the joints and fractures of the bedrock. At the time of the study, only diamond core borings CN-CH-1, -9, and -11 were open and capable of being photographed.

(3) Investigations in 1988 were conducted in March and May and consisted of geophysical logging and another borehole photography study. Geophysical logging was performed by Water Resources Division, U.S. Geological Survey (USGS), Albuquerque, New Mexico. Diamond core borings CN-CH-1, -3, -9, -11, -12, and -17 through -25 were studied using Natural Gamma, Gamma Gamma Density, Neutron, and Caliper logs. Southwestern Division (SWD) Laboratory, U.S. Army Corps of Engineers conducted the borehole photography study in diamond core borings CN-CH-1, -3, -9, -15 through -18, and -20.

(4) Investigations in 1989 were conducted in March and consisted of a third borehole photography study. SWD Laboratory conducted the study in diamond core borings CN-CH-19 through -23, -26, -29, and -30, and Denison boring CN-D-1.

3.02 Materials Testing Prior to Construction.

a. Rock Testing. A total of 21 rock core samples from the RCC dam area and 15 rock core samples from the original spillway area were tested for unconfined compressive strength, bulk specific gravity, and modulus of elasticity. Testing of core samples from diamond core borings CN-CH-1 through -3 was performed by Fox and Associates of New Mexico, Inc. Samples from CN-CH-10 through -12 and -15 through -22 were tested by SWD Laboratory. Test results are summarized in Paragraph 3.03i and included in Appendix E.

b. Soil Testing. Sieve analyses, moisture, and Atterberg limits tests of material excavated from trenches CN-T-8 through -13 were performed by Fox and Associates of New Mexico, Inc. in 1984. Sieve analyses and Atterberg limits tests of material excavated from trenches CN-T-14 through CN-T-20 were performed by Western Technologies, Inc. in 1987. Test results are summarized in Paragraph 3.03j and contained in Appendix E. No strength or consolidation tests were performed on this material.

c. Aggregate and Riprap Testing. Two bulk samples (one natural sand and gravel sample and one 100-pound bulk chunk stone sample) were submitted to SWD Laboratory for testing of concrete aggregate and riprap. Petrographic and sieve analyses, were performed on the bulk aggregate sample. Separate gradation, specific gravity, absorption, and soundness (magnesium sulfate) tests were performed on the plus #4 and minus #4 material. In addition, particle shape and LA abrasion tests were performed on the plus #4 material, and the minus #4 material was tested for organic impurities. The riprap sample, composed of site limestone, was subjected to specific gravity,

absorption, LA abrasion, and soundness (magnesium sulfate, freezing and thawing) tests. Test results are summarized in Paragraph 3.03K and included in Appendix E.

d. Water Pressure Tests. Water pressure tests were performed on diamond core and air hammer borings CN-CH-9 through -12, CN-CH-15 through -23, and CN-CH-25 through -32. Upon completion of drilling, each boring was tested using a single inflatable packer set at depths where it could properly seal the boring. The interval between the bottom of the packer and the bottom of the hole (BOH) was then tested. All pressure testing was performed using water, with maximum pressures based on one pound per square inch (psi) of pressure per foot of depth to the top of the packer. In the majority of tests, the design pressure could not be obtained. Tests were performed for approximately ten minutes, with accurate water use recorded during each test. The results of water pressure tests are discussed in Paragraph 3.03e and contained in Appendix E.

3.03 Results of Pre-construction Explorations, Investigations and Tests.

a. RCC Dam and Appurtenant Structures - Explorations. Two potential dam alignments were investigated, one along a line connecting CN-CH-1, CN-CH-2, and CN-CH-3, and another along a line connecting CN-CH-16 and CN-CH-18. The final alignment was located between these two. During the subsurface explorations, little overburden was found in the borings located near the abutments for the proposed dam alignment. All borings were vertical. The major rock types encountered were limestone, argillaceous limestone, and shale. Other than areas of poor quality rock to be removed during foundation preparation, the bedrock was considered to be a suitable foundation for the dam. Plates No. A-7 and No. A-8 in Appendix A contain a geologic cross section along the dam axis.

(1) On the left abutment, in CN-CH-1, unconsolidated Palomas gravel was 3.4 feet thick. Consolidated Mud Mountain fanglomerate was encountered at 3.4 feet, with limestone encountered from 5.0 feet to 100.5 feet (BOH). Several thin interbedded shale layers were located within the limestone, with one layer extending from 90.1 feet to 96.4 feet.

(2) On the right abutment, slopewash overburden was less than 0.5 feet thick in both CN-CH-3 and -18. CN-CH-3 was drilled into hard limestone to 35 feet, below which interbedded shale and siltstone was encountered to 37.5 feet. The remainder of the boring was drilled into hard limestone except for two shale layers encountered from 57.5 feet to 63.3 feet and from 77.4 feet to 79.0 feet (BOH). CN-CH-18 was drilled approximately 65 feet upstream of CN-CH-3 into hard limestone and argillaceous limestone except for shale encountered between 71.8 feet and 74.0 feet and weathered shale between 86.5 feet and 96.4 feet. Highly weathered and solutioned zones were encountered between 20.6 feet and 24.1 feet and between 35.1 feet and 37.8 feet. Because the shale and weathered zones were encountered at relatively greater depths and could be grouted if necessary, it was assumed that they would not present a problem during or after construction.

(3) CN-CH-2, -15, -16, and -28 were drilled in the channel bottom. CN-CH-2 and -16 were drilled along the proposed alignments, CN-CH-15 was drilled at the stilling basin, and CN-CH-28 was drilled at the low level outlet works intake tower. Alluvium overburden was 36.0 feet thick in CN-CH-2, 31.0 feet thick in CN-CH-15, 22.2 feet thick in CN-CH-16, and 2.5 feet thick in CN-CH-28. The majority of rock encountered was hard limestone and argillaceous limestone with beds of shale similar to those encountered in the abutment borings. The upper 10 feet of limestone in CN-CH-16 contained numerous solutioned and weathered zones similar to those encountered in CN-CH-18.

b. Earth Embankment Section - Explorations. Because of the excessive depths to suitable foundation rock in this area, the construction of an earth embankment was considered appropriate. The earth embankment foundation was explored by CN-CH-5, -5a, -6, -7, -7a, and -17. CN-CH-5a and -7a were drilled adjacent to CN-CH-5 and -7, respectively, in order to verify depth to bedrock as determined by the surface seismic refraction survey. Unconsolidated Palomas gravel overburden encountered was 31.0 feet thick in CN-CH-5, 101.0 feet thick in CN-CH-6, 46.0 feet thick in CN-CH-7, and 22.4 feet thick in CN-CH-17. In addition, there was 5.2 feet of consolidated Mud Mountain fanglomerate underlying the Palomas gravel in CN-CH-17. Otherwise, bedrock consisted of shale and minor limestone in CN-CH-5 and CN-CH-6, limestone in CN-CH-7, and limestone and shale in CN-CH-17.

c. Original RCC Spillway - Explorations. The original proposed project included a RCC spillway to be located in an adjacent canyon to the south of the dam. This site was explored in 1986 and early 1988. After concerns were raised regarding the quality of the foundation rock, additional explorations were performed in late 1988 and early 1989. Due to the significant amount of solutioning and alteration encountered in these borings, it was determined that this site would not provide an acceptable foundation for the auxiliary spillway.

(1) Fifteen diamond core borings, one auger boring, and one Denison boring were drilled in the area of the spillway. The Denison boring was drilled for the borehole photography study, and the auger boring was drilled to determine the suitability of excavated material for use as aggregate.

(2) CN-CH-9, -23, and -27 were drilled at the left abutment of the spillway. Overburden consisted of 4.8 feet to 9.8 feet of gravelly sand with cobbles. In CN-CH-9, the remainder of the boring encountered hard interbedded limestone and argillaceous limestone to 126.0 feet. CN-CH-23 encountered hard interbedded limestone and argillaceous limestone to 100.5 feet, with weathered solutioned zones from 26.1 feet to 29.5 feet and from 53.0 feet to 56.7 feet. CN-CH-27 encountered hard limestone to a depth of 50.4 feet.

(3) CN-CH-11, -21, -22, and -26 were drilled at the right abutment of the spillway. Overburden consisted of 7.6 feet to 24.8 feet of silt, sand, gravel, and scattered cobbles. CN-CH-11 encountered hard limestone interbedded with minor argillaceous limestone to a depth of 126.0 feet. A brecciated and solutioned shear zone was encountered between 108.0 feet and 112.1 feet. CN-CH-21 encountered limestone with abundant shale beds to a

depth of 137.4 feet. Numerous weathered and solutioned zones were encountered throughout the boring. CN-CH-22, which had the thickest overburden, encountered limestone, argillaceous limestone, and minor shale to a depth of 101.0 feet. A few relatively thin weathered and solutioned zones were also encountered throughout the boring. CN-CH-26 encountered hard interbedded limestone and argillaceous limestone to a depth of 98.1 feet. Numerous weathered and solutioned zones were encountered throughout the boring.

(4) CN-D-1 and CN-CH-10, -12, -19, -20, -25, and -29 were drilled along the valley of the spillway between approximately 300 feet upstream of the spillway alignment and 200 feet downstream of the spillway alignment. All borings encountered limestone, argillaceous limestone, and moderate to abundant amounts of shale. Zones of complete weathering and/or solutioning were encountered in all the borings, but were somewhat less pronounced in CN-CH-10 and -12 upstream of the alignment. The top 50 feet of bedrock in CN-CH-19 was severely weathered and solutioned. Overburden thickness varied between 4.8 feet in CN-CH-10 to 19.0 feet in CN-CH-19. Overburden consisted of alluvium composed of silt, sand, and gravel with minor clay and scattered cobbles and boulders.

d. Auxiliary Spillway - Explorations. In April 1989, it was determined that the bedrock at the original spillway site was unsuitable for foundation purposes, and the project design was changed to include a RCC dam overflow section and a RCC stepped auxiliary spillway to be located at the site of the approach channel for the original spillway. Explorations were conducted at the site in 1984, 1986, 1987, and 1988 to determine depth to bedrock, quality of bedrock, suitability of material for use as fill or aggregate, and determination of excavation techniques for the approach channel. In 1989, additional explorations were conducted to determine quality of and depth to bedrock for the foundation for the auxiliary spillway. It was determined that the materials encountered would provide a suitable foundation for the spillway and chute toe. A well consolidated conglomerate within the Santa Fe group was encountered in CN-CH-40 through -45. It was determined that this conglomerate, if widely distributed, would provide a suitable foundation for the auxiliary spillway. Plates No. A-9, No. A-10, and No. A-11 in Appendix A contain geologic cross sections along the ogee axis of the auxiliary spillway.

(1) Diamond core borings CN-CH-4 and -24 were drilled on the left abutment of the auxiliary spillway. Overburden, consisting of colluvium and fanglomerate composed of sand, gravel, and minor clay, was 59.0 feet thick in CN-CH-4 and 35.1 feet thick in CN-CH-24. Both borings encountered hard limestone with minor shale below the overburden.

(2) Trenches CN-T-14 through -20 were excavated in 1987 to depths ranging from 2.0 feet in CN-T-20 to 6.0 feet in CN-CH-14. Auger borings CN-8A-01 through -7 were drilled in 1988 to depths ranging from 8.0 feet in CN-8A-07 to 39.0 feet in CN-8A-03. The purpose of these explorations was to determine the classification of materials to be excavated for the approach channel to the original spillway, the excavation techniques, and the suitability of this material for use as random fill and/or aggregate. CN-CH-13 and -14 were drilled to 50.0 feet each to determine depth to bedrock. CN-CH-13 encountered

limestone at 48.0 feet and CH-CN-14 remained in overburden.

(3) Diamond core borings CN-CH-32 through -45 were drilled in various areas of the auxiliary spillway to determine the depth to, and quality of, the Paleozoic bedrock formations. Unconsolidated Palomas gravel varied in thickness between 20.5 feet in CN-CH-36 and 74.0 feet in CN-CH-43. A well indurated, hard conglomerate of the Santa Fe group was encountered at a depth of 63.4 feet in CN-CH-40, 36.5 feet in CN-CH-41, 59.5 feet in CN-CH-42, 74.0 feet in CN-CN-43, 46.0 feet in CN-CH-44, and 45.0 feet in CN-CH-45. The thickness of the conglomerate varied between 10.9 feet and 64.0 feet. The remaining borings did not encounter the conglomerate. The conglomerate disconformably overlies hard limestone with minor argillaceous limestone and shale with zones of severe weathering and/or solutioning. The Palomas gravel was overlying the limestone, argillaceous limestone, and shale in the borings which did not encounter the conglomerate.

e. Water Pressure Tests. Water loss during water pressure tests ranged between 0.11 cubic feet per minute (CFM) in CN-CH-16 and 23.0 CFM in CN-CH-20. The Feature Design Memorandum states, "Based on results of these tests, as well as observation of the core, it was assumed that permeability is controlled primarily by near vertical jointing in the bedrock, resulting in very low horizontal permeability."

f. Borehole Photography Studies. The results of the borehole photography studies were inconclusive. The borehole camera did detect some weathered and solutioned zones but was less successful in providing information on strike and dip of joints. Two well defined joint sets (Sets 1 and 2) and one potential joint set (Set 3) were recognized with orientations as follows:

	Dip Direction	Dip
Set 1	270 +- 25 degrees	70 +- 10 degrees
Set 2	60 +- 30 degrees	25 +- 15 degrees
Set 3	180 +- 40 degrees	70 +- 20 degrees

Set 2 likely consists of bedding features that may actually not be joints. Surface observations revealed at least two other likely sets of steeply dipping joints not detected in the studies. Because all borings were vertical, it is probable that many of these joints were not encountered. A detailed surface joint survey was not conducted.

g. Geophysical Logging. "A Summary of Cuchillo Negro Geophysical Logs" was provided by the USGS and is included with the logs in Appendix E. Lithology, as determined from interpretation of the geophysical logs, closely correlated with the physical logs. The USGS report is summarized as follows: Interpretation of the geophysical logs concluded that fracturing was generally random and without direct communication between boreholes. Due to the approximate 25 degree dip and the block-like nature of the formation, it was concluded that groundwater would have both vertical and horizontal components, and the areal permeability, therefore, would probably be low. This interpretation and the conclusions provided by the USGS do not seem to fit

what was observed during construction. In actuality, random fracturing would increase the communication in general, and the block-like nature would provide additional flow paths, resulting in a relatively high areal permeability, assuming that joints were open.

h. Seismic Refraction. Results of the seismic refraction survey indicate the presence of five distinct P-wave velocity layers. One of the velocity layers was the bedrock surface, which was dipping parallel to the dip of beds observed at the surface. Other layers correspond to surface soil, unconsolidated Palomas gravel, consolidated Palomas gravel, and the Santa Fe group. Only the surface soil and unconsolidated Palomas gravel were encountered during excavation of the inspection trench.

i. Rock Testing. Unconfined compressive strength tests for core samples from the dam and spillway yielded strengths of 10,000 psi to 25,000 psi for limestone samples, and 4,000 psi to 10,000 psi for calcareous shale samples. Altered limestone was not tested. These results indicated that the foundation rock was suitable for RCC or earth and rockfill dams, provided the weathered argillaceous limestone and shale zones exposed near the surface were excavated.

j. Soil Testing. Results of sieve analyses and Atterberg limits tests indicated that the material excavated from the auxiliary spillway would be suitable for use as semi-impervious and random fill for the earth embankment section, and that the material from the borrow area would be suitable for use as aggregate for RCC and conventional concrete.

k. Riprap and Aggregate Testing. The aggregate and bulk limestone samples submitted for chemical and physical deterioration tests were considered of adequate quality for use as concrete aggregate and stone protection, respectively.

3.04 Investigations During Construction. Additional investigations were conducted in May 1990 at the left abutment of the RCC dam in two locations: 1.) along the alignment of the high level outlet works to determine the quality of foundation material, and 2.) at the far left end of the foundation trench to determine depth to bedrock. Investigations consisted of obtaining undisturbed record samples and grab samples of joint infilling material, weathered shale, and sheared breccia/clay and conducting subsurface explorations.

a. Subsurface Explorations. Subsurface explorations consisted of one Denison boring on the left abutment from the intermediate bench excavated as part of the modification to the left abutment foundation trench, four air percussion borings along the centerline of the high level outlet works, and two air percussion borings along the dam axis in the left abutment foundation trench. Logs of the Denison boring and the HLOW air percussion borings are located in Appendix B. Table 3-2 provides pertinent data regarding the subsurface explorations. Plate No. A-14 in Appendix A is a plan view illustrating the location of explorations during construction.

Table 3-2 Construction Explorations Pertinent Data

Number	Type	Depth in Feet	Location	Elev feet	Date Completed
PH-0	Air Percussion	42.0	HLOW STA 1+85U	4674	May 1990
PH-1	Air Percussion	45.0	HLOW STA 2+25U	4673	May 1990
PH-2	Air Percussion	60.0	HLOW STA 2+75U	4672	May 1990
PH-3	Air Percussion	32.0	HLOW STA 3+25U	4670	May 1990
PH-4	Air Percussion	25.0	Lft Abtmnt STA 0+25C	4706	May 1990
PH-5	Air Percussion	11.0	Lft Abtmnt STA 0+65C	4706	May 1990
C-37-1	8-inch Denison	12.0	Lft Abtmnt STA 3+50D	4655	Dec 1990

(1) Drilling of the air percussion borings was performed by McCaw's Drilling and Blasting, Inc., the drilling and blasting sı contractor for the project. The original plan was to drill diamond core borings using the Contractor's QC subcontractor, Western Technologies, in addition to the air percussion borings. This firm submitted a bid in excess of $300.00/LF for diamond core drilling, well above any possible negotiated amount. A scope of work was developed, and a delivery order was negotiated with an A-E contractor to perform the drilling starting in late June or early July of 1990. The A-E Contractor's drill rig was involved in a roll-over accident shortly before drilling was to commence, and the delivery order was canceled. By that time, the air percussion holes had been completed, as well as the foundation drilling and blasting for excavation. It was felt that additional explorations were no longer necessary because the data provided by the air percussion drilling was adequate.

(2) Drilling of the Denison boring was performed by Fort Worth District, U.S. Army Corps of Engineers (SWF) in December 1990. A number of borings were originally planned to be drilled from an intermediate bench being excavated on the left abutment. The purpose of the borings was to obtain undisturbed samples of the shale and shear zones encountered during excavation of the foundation trench. Only one boring was completed, and a satisfactory sample was not obtained. The Denison barrel was not capable of penetrating the dipping contact between sound limestone and overlying shale due to deviation of the bit along the contact and subsequent binding of the barrel against the side of the vertical boring. The drill rig was not capable of angle drilling. It was determined that record push sampling would be attempted later.

b. <u>High Level Outlet Works - Explorations.</u> In March and April of 1990, excavation of the foundation trench on the left abutment of the RCC dam exposed a zone of highly fractured and weathered limestone, nodular shale, shale, and sheared shale between the high level outlet works and the low level outlet works. The top of this zone was at an elevation of 4,670 feet (NGVD), and the bottom of the zone was at an elevation of 4,648 feet (NGVD). The base of this zone consisted of a sheared breccia 1 foot to 3 feet thick. Below this zone was 10 to 20 feet of limestone that had been severely solutioned along joints. Additional explorations were undertaken to determine how far

back into the abutment this zone extended and to recognize any stratigraphic or structural features related to its formation.

(1) Four air percussion borings were drilled in May 1990 from the excavated high level outlet works bench at an elevation of 4,674 feet (NGVD). The borings were drilled along the centerline of the outlet works from 32.0 feet to 60.0 feet deep. All borings encountered fractured and weathered limestone, argillaceous limestone, nodular shale, shale, and siltstone. Thin brecciated and/or sheared zones were detected throughout. Borings PH-0 and PH-1 encountered a reddish brown shear-zone clay at 41.0 feet and 44.0 feet, respectively. Boring PH-2 encountered the shear zone between 44.0 feet and 55.0 feet, and hard micritic limestone between 55.0 feet and 60.0 feet (BOH). PH-3 was the first boring drilled and was drilled at an angle of 25° from vertical (normal to bedding). The air ducts in the drill bit became clogged and the bit and steel wedged in the boring at a depth of 32.0 feet. The remaining borings were drilled vertically.

(2) Interpretation of the logs from these borings, combined with observations made of the exposed foundation, indicated that both the zone of poor quality rock and the underlying shear zone were stratigraphically controlled and would not diminish with continued excavation of the foundation trench. It also appeared that the solutioning of the underlying limestone was structurally controlled. The solution features developed along a closely spaced set of nearly north-south striking, steeply dipping joints that had experienced some movement. It was determined that laying back and benching of the left abutment foundation trench slope would allow for proper removal and/or treatment of unsatisfactory foundation materials from the solutioned limestone.

c. Left Abutment - Explorations. Using the Contract drawings, the Contractor excavated the left abutment foundation trench for the RCC dam between STA 0+14C and STA 1+16C to an elevation of 4,706 feet (NGVD). This elevation was indicated on the Contract drawings to be four to five feet below top-of-rock. After excavation, bedrock was not exposed between STA 0+14C and STA 1+02C. Two air percussion borings were drilled along the dam axis in this area to determine the depth to bedrock. All earlier explorations and results of the seismic refraction survey indicated that the bedrock surface followed bedding dip, which is approximately 25° to the northeast. It is likely that the bedrock surface shown on the contract drawings was a drafting error not detected during review of the documents.

(1) Two air percussion borings were drilled in May 1990 along the centerline axis of the dam. PH-4 was drilled at STA 0+25C and encountered limestone at a depth of 20.5 feet. From 0.0 feet to 18.5 feet. overburden consisted of colluvium. Consolidated Mud Mountain fanglomerate was encountered between 18.5 feet and 20.5 feet. PH-5 was drilled at STA 0+65C and encountered limestone at 10.5 feet. From 0.0 feet to 8.5 feet, overburden consisted of unconsolidated fanglomerate. Consolidated fanglomerate was encountered from 8.5 feet to 9.5 feet. There was one foot of clay on top of the limestone which was probably the remnant of a clay-filled joint.

d. Materials Sampling and Testing. Seven undisturbed and four disturbed samples were taken of material encountered during excavation of the foundation trench on the left abutment of the RCC dam. Three disturbed grab samples, FS#1 - FS#3, were taken from fracture infilling material, and one, FS#4, was taken of the shale and shear/breccia zone. Undisturbed samples, designated with the suffix L, were collected by using the bucket of the excavator to hydraulically push 8-inch-diameter by 12-inch-long record sample tubes into in-situ material. Five undisturbed samples were taken of the shale and shear/breccia zone, and two undisturbed samples were taken from material filling the large solution cavities in the underlying limestone. All samples were subjected to sieve analysis, moisture, and Atterberg limits tests. In addition, five undisturbed samples were subjected to consolidation and/or direct shear tests. Testing was performed by Southwestern Division Laboratory. Table 3-3 provides pertinent information regarding the samples. The laboratory report of test results is located in Appendix E.

Table 3-3 Sampling Pertinent Data

Sample No.	SWD Sample	Material Sampled	elev (feet) (NGVD)	Location along dam alignment
FS #1	1	Composite of joint filling	4650	1+35D - 2+00D
FS #2	2	Composite of shear/breccia	4640	1+50D
FS #3	3	Composite of cavity filling	4635	1+35D - 1+60D
FS #4	4	Composite of joint filling	4670	2+70D
21L	5	Shale and shear zone	4640	1+35D
17L	6	Shale and shear zone	4640	1+35D
7L	7	Shale and shear zone	4640	1+35D
30L	8	Shear/breccia zone	4638	1+50D
27L	9	Clay/shale/shear zone	4638	1+55D
24L	14	Cavity infilling	4630	1+60D
16L	15	Cavity infilling	4635	1+60D

3.05 Results of Investigations During Construction.

a. Subsurface Explorations. Drill logs of cuttings from borings PH-0 through -3 indicate that the limestone, nodular limestone, shale, and sheared and brecciated zones encountered during excavation of the foundation trench are stratigraphic features that extend for an undetermined distance into the left abutment. The degree to which the underlying limestone has been solutioned is dependent on the spacing and amount of movement along joints. All borings encountered the shear zone and underlying solution cavity infilling material. The exposed solution zone had developed along a closely spaced set of high angle joints with evidence of movement that was more susceptible to solutioning. This solution zone did not seem to be as well developed deeper into the abutment as that exposed in the excavation. Other field geological observations indicate that a similar zone may exist approximately 80 feet behind the present limits of excavation. Because this zone is stratigraphically and structurally controlled, it was determined that

more suitable foundation material would not be found with additional excavation, and the foundation would have to be prepared as well as possible with the materials present. A new cut slope and intermediate bench was deemed necessary, using more tightly controlled blasting in order to provide a foundation surface that could be properly prepared for RCC placement.

(1) Borings PH-4 and -5 indicate that the bedrock surface on the left abutment is dipping at approximately the same angle as bedding, as would be expected. These results repeat what had already been determined by the preconstruction investigations. The contract drawings erroneously show a nearly horizontal surface. It was determined that the Mud Mountain fanglomerate, where consolidated, would provide a suitable foundation, and this area would not have to be excavated to the limestone, therefore reducing the amount of extra excavation and subsequent RCC that would be required in this area.

b. Materials Testing. All samples could be easily broken down in order to perform soils tests. Results of the sieve analyses, moisture, and Atterberg limits tests on the shale, shear, breccia, and clay infilling material indicated that all material sampled contained greater than 90% fines, was classified as CH, and was described as shale or fat clay. Moisture content ranged between 8.8% and 37.7% Results of direct shear and consolidation tests on the undisturbed samples are in Appendix E and indicate that the material is normally consolidated. Shear strengths on these samples ranged from c=0, o=10.2° to c=0, o=17°. Based on these results, it was determined that the material would not be susceptible to piping and would be suitable for the foundation. This material, however, was removed and/or the surface was laid back during foundation preparation, and the resulting voids were filled with dental concrete prior to receipt of test results. Consolidation tests were also performed on the sheared breccia material. Although overburden pressures used during testing do not reflect the actual pressures to which the material had previously been subjected, adjustments to the consolidation curves were made to reflect initial overburden pressures, and conservative values were used to compute possible consolidation of the brecciated zone. The computations indicate that a settlement of from one to two inches is possible due to the consolidation of the brecciated zone. This settlement could cause minor cracking but would not pose a threat to the structure. The control joint at station 2+10C was constructed to allow for this settlement.

IV - SPECIAL DESIGN CONSIDERATIONS

4.01 General: Unanticipated geologic conditions encountered during excavation of the foundation trench for the RCC dam and the foundation for the auxiliary spillway resulted in changes in the project design. Significant modifications were made to the design of the RCC dam, high level outlet works, and auxiliary spillway. The geologic conditions and the changes made to the design are discussed in greater detail in Section 2.06, Unusual or Unanticipated Geologic Conditions Encountered During Construction. Design and construction details of these modifications as well as final excavation sections are in the "As Built" structural drawings provided by the Contractor, which are included as Plate No. A-18 through Plate No. A-37 in Appendix A.

4.02 RCC Dam. A major change in the design of the RCC dam was required to compensate for concerns regarding stability of the structure as a result of the unanticipated geologic conditions encountered during excavation of the foundation trench for the left abutment.

a. Design personnel felt that the shear/breccia zone described in Paragraph 2.04a(3)(v) could provide a weak slippage surface that could result in a large block of bedrock sliding into the left abutment or in a downstream direction. Other observed conditions, however, preclude the likelihood of such an event. The bedding dip direction of the strata and shear/breccia zone is toward the left abutment. A large block of foundation bedrock would not be able to displace any in-situ material by sliding down-dip along a possible slippage surface parallel to bedding. Horizontal movement in the downstream direction would also be difficult. As described in Paragraph 2.04a(2), the strata is gently folded normal to the direction of the stream channel here. This folding would provide an irregular surface which would impede horizontal slippage.

A structural stability analysis was performed by Boyle Engineering Corporation, the design A-E firm for the project. This analysis assumed that a vertical, smooth joint existed in the foundation extending through the entire section between the two outlet works and striking normal to the axis of the dam. In addition, although the formation dip was assumed, the folding was not. This resulted in a relatively conservative analysis of the potential for major slippage in the left abutment. The factor of safety was calculated to be 2. The stability analysis with all calculations is included in Appendix E.

b. Additional concerns for structural stability were also raised with regards to the type of foundation materials present in the left abutment. The shear/breccia zone and the shale and siltstone units are described in Paragraphs 2.04a(3)(v) and 2.06a(1) and are not present on the right abutment. These units were potentially weaker and more susceptible to compression than the limestone underlying them. Differential settlement of the foundation on the left abutment could result from consolidation of these units due to the weight of the structure itself. Test results, as discussed in Paragraph 3.05b

and included in Appendix E, indicate that the shale and shear/breccia zone material sampled are of sufficient strength to withstand the load of the structure. However, after consultations with Southwestern Division, Headquarters, and design geotechnical personnel, it was determined that two control joints would be installed through the width of the RCC dam, one at STA 2+10C, and another at STA 3+72C (photo 42). A modification to the contract was issued to install the control joints. This added $229,661.00 to the cost of the contract.

4.03 High Level Outlet Works (HLOW). As described in Paragraph 2.06a(1)(i)(b), the right side of the HLOW channel downstream of the box culvert section was intended to be in-situ bedrock. This material was highly fractured and of less width than anticipated. In addition, the narrow foundation could not support the trashrack at the intake to the HLOW. As a result, the contract was modified to provide a concrete "U-shaped" channel between STA 2+37U and STA 3+12U (photo 33), and the trash rack was moved to the upstream face of the dam, where the foundation bench was of sufficient width (photo 43). These modifications added $95,085.00 to the cost of the contract.

4.04 Auxiliary Spillway.

a. A consolidated conglomerate within the Palomas gravel was encountered during excavation of the spillway. This material could not be ripped using equipment specified in the contract (photo 38). It was decided to found the chute toe of the spillway in this conglomerate. The spillway design was modified to move the chute toe from an elevation of 4,628 feet, to an elevation of 4,687 feet. This eliminated a significant amount of RCC and the need for fill above the chute toe.

b. A field modification was initiated to provide a cut-off wall on the right abutment of the auxiliary spillway. The original design did not provide for control of seepage along the RCC-foundation interface which would occur during high pool. A near vertical trench was excavated on the right abutment approximately three feet into the foundation material (photo 44). This trench was filled with RCC as each lift of the RCC slope protection was placed. The intent was to provide at least some minimal positive cut-off to potential seepage.

V - EXCAVATION

5.01 General. The prime contractor for the construction of the embankment, spillway, and outlet works was PCL Civil Constructors, Inc. of Phoenix, Arizona. Drilling, blasting, and the majority of rock bolt installation was performed by PCL's subcontractor, McCaw's Drilling USA, Inc., also of Phoenix, Arizona with corporate offices in Alberta, Canada. Construction began on 16 November 1989 with the clearing and stripping for haul roads and the project office/laboratory complex. The Excavation Plan submitted by the Contractor is included in Appendix F. Plates No. A-16 and A-17 in Appendix A illustrate the as-built foundation excavation plan for the RCC dam and auxiliary spillway respectively.

5.02 Excavation Grades. As-built excavation grades varied from the designed grades for the RCC dam foundation trench, the earth embankment inspection trench, and the auxiliary spillway cut. This variation was primarily the result of the unusual and unanticipated geologic conditions described in Section 2.06. Plate No. A-15 in Appendix A is a cross section along the dam axis that compares the designed versus as-built excavation slopes.

a. RCC Dam Foundation Trench. The as-built excavation grades in the foundation trench varied significantly from the as-designed grades. Presplit blasting was required for all rock excavation involving final slopes of 1V on 1H or greater. Many final presplit faces were very irregular. This was the result of foundation conditions. Most presplit faces were parallel or nearly parallel to the three steeply dipping joint sets described in Paragraph 2.04a(1) causing the rock to "fall off" along those joints shortly after presplitting. In addition, there were areas of extreme variation in the degree of weathering, consolidation, and hardness of the bedrock which contributed to some of the highly irregular nature of many presplit faces. In areas where the foundation consisted of competent, slightly jointed limestone, the presplitting worked well, and final slope faces were well within the tolerances specified.

(1) The foundation trench floor surface was highly irregular, resembling stair steps as described in Paragraph 2.06a(2)(ii). This was the result of bedrock breaking along the steeply dipping joints and the bedding plane joints described in Paragraph 2.04a(1).

(2) A 70-foot-thick section of bedrock exposed on the left abutment contained highly jointed limestone beds, shale beds, and a shear/breccia zone. This interval is described in detail in Paragraphs 2.04c(2) and 2.06a(1). The removal of this material required benching of the slope between the high level outlet works and the low level outlet works. The resulting slope was a major deviation from the as-designed slope.

(3) In situ bedrock was originally intended to form the right side of the discharge channel for the HLOW and provide forming for the concrete encasement of the conduit for the LLOW. This material proved unsuitable for its intended

function and was removed as described in Paragraphs 2.06a(1)(i)(b) and 2.06a(1)(ii), resulting in a deviation from the as-designed excavated slopes.

(4) The far left side of the foundation trench between STA 0+00C and STA 0+95C was excavated to depths in excess of those shown in the Contract Plans. This was due to an error in the drawings as described in Paragraph 3.04c.

(5) Removal of the clay infilling of joints and the altered material described in Paragraphs 2.04c(1) and 2.04c(2) resulted in a highly irregular surface in sections of the foundation trench on both the left and right abutments (photos 21 & 36).

b. Earth Embankment Inspection Trench. The earth embankment inspection trench was designed to be 10 feet wide at the bottom with cut slopes of 1V on 1H to the ground surface. The as-built width was 15 feet. The width was increased in order to accommodate excavation and placement utilizing the Contractor's Caterpillar Model 631D earth scrapers (photo 45).

c. Auxiliary Spillway. The as-built excavated slopes for the auxiliary spillway varied significantly from the as-designed slopes. The as-built chute toe was relocated to an elevation of 4,687 feet (NGVD) from the as-designed elevation of 4,628 feet (NGVD). This was the result of the Contractor encountering a consolidated conglomerate classified as rock during excavation. The change in the designed excavation resulting from the conglomerate encountered is described in Paragraphs 2.06b(1) and 4.04.

5.03 Dewatering & Care of Water. The Contractor submitted a Dewatering Plan as specified in the contract. This plan is included in Appendix F. The Specifications required the Contractor to design and install a system sufficient to divert Cuchillo Creek through the lower outlet works. "The diversion shall be a system of cofferdams, temporary culverts or ditches sufficient to divert 250 cubic feet per second (cfs) through the lower outlet works. The cfs will be measured at the USBR gaging station upstream from the project. The location of the gage is shown on the drawings. If the stream flow exceeds 250 cfs, and the flow in excess of 250 cfs causes damage to permanent construction or to previously prepared foundations, an equitable adjustment will be made under the CHANGES clause of the Contract Clauses."

a. There were two flaws in the "Dewatering and Care of Water" clause in the specifications: 1) There was no method specified for diverting water prior to construction of the LLOW. 2) The USBR gaging station specified was actually a high water level alarm with no capability of actually measuring flow volume in the creek.

b. The Dewatering Plan submitted by the Contractor was not adhered to, primarily due to the less-than-expected amount of shallow groundwater which was encountered during most of the construction. The storage pond upstream of the dam on the right side of the channel and the collection line from the existing well were not constructed. The existing well was not capable of producing a sufficient volume of water for construction purposes. A new well described in Paragraph 2.04e was also inadequate. An existing well located

one mile upstream of the project was utilized to provide an adequate supply of construction water. The Contractor utilized a pump and one discharge line to dewater the foundation trench after flooding. By the time actual construction began, groundwater levels had dropped, and dewatering was not a problem. The Contractor did not have to install the upstream and downstream pumps and discharge lines described in the Dewatering Plan. The temporary diversion dam described in the dewatering plan was constructed once the conduit for the LLOW was in place. No flows in the channel, however, were recorded during construction.

(1) The Contractor constructed a diversion channel with levees along the creek bed through the area of the RCC batch plant and aggregate stockpiles. The diversion channel emptied into a temporary storage pond (photo 46). Using the Chezy-Manning equation, the Contractor calculated that this diversion channel had a capacity in excess of 330 cfs. These calculations are included with the dewatering plan in Appendix F. On 31 July 1990, the channel was breached, flooding the aggregate stockpiles (photo 47) and the foundation trench, completely refilling the excavation with silt and sand (photos 26 & 27). Modification No. P00046 was issued to repair flood damage to the LLOW. This modification added $77,000.00 to the cost of the contract.

(2) The Contractor recorded 15 flooding events between 14 July 1990 and 2 November 1990. As calculated by the contractor, 11 of these events were the result of flows in excess of 250 cfs. Table 5-1 is a list of these events as determined by the Contractor. The Contractor filed a claim for flood damage from flows in excess of 250 cfs for $156,017.23. Modification No. P00047 was issued to settle the claim for $71,696.

Table 5-1 Summary of Flood Events in Cuchillo Negro Creek

Date	Estimated Flow Rate	Estimated Duration
14 July 1990	340 cfs	3 Hrs
22 July 1990	900 cfs	5 Hrs
27 July 1990	900 cfs	6 Hrs
31 July 1990	4,300 cfs	7 Hrs
4 August 1990	100 cfs	4 Hrs
13 August 1990	300 cfs	10 Hrs
16-17 August 1990	315 cfs	30 Hrs
21 August 1990	300 cfs	12 Hrs
24 August 1990	75 cfs	18 Hrs
27-28 August 1990	3,000 cfs	12 Hrs
5-6 September 1990	4,100 cfs	70 Hrs
20-21 September 1990	1,080 cfs	18 Hrs
22-24 September 1990	5,000 cfs	60 Hrs
2-3 October 1990	100 cfs	16 Hrs
2 November 1990	100 cfs	2 Hrs

5.04 Overburden Excavation. Areas requiring overburden excavation included the channel bottom at the RCC dam section, the majority of the auxiliary spillway, the office/laboratory site, access and maintenance/haul roads, and the inspection trench for the earth embankment.

a. Although the Palomas gravel is classified as bedrock as described in Paragraph 2.04b, the majority of this unit encountered during excavation of the earth embankment inspection trench and the auxiliary spillway was removed as common (overburden) excavation. The contractor utilized the following equipment to perform common excavation:

2 ea Caterpillar D9H Bulldozers
2 ea Caterpillar D8K Bulldozers
4 ea Caterpillar 631D Scrapers
1 ea International Harvester Transtar 4300 Water Truck
5 ea Caterpillar 769 End Dump 35-Ton Dump Trucks

This equipment was effectively utilized, well maintained, and adequate for the common excavation required (photo 48).

b. Slope support was not required for any of the common excavation. The excavated slopes were stable as designed, as a result of both the slope angle and the slightly consolidated nature of the overburden material.

5.05 Rock Excavation. For all excavation classified as rock excavation, the Contractor was required to excavate using controlled blasting techniques. Blasting operations were performed in accordance with the applicable provisions of the Corps of Engineers, Manual EM-385-1-1, dated 1 April 1981, revised October 1987, titled "Safety and Health Requirements Manual." Areas requiring blasting included the left abutment and chute toe of the auxiliary spillway, the main RCC dam section foundation trench, the high and low level outlet works' foundations, and a portion of the access/maintenance road. Shot rock removal was accomplished as described in the excavation plan submitted by the Contractor. The Contractor utilized the following equipment to remove shot rock:

1 ea Caterpillar 988B Front-end Loader
1 ea Caterpillar 245/235 Trackhoe Excavator
1 ea Caterpillar D9H Bulldozer
5 ea Caterpillar 769 End Dump 35-Ton Dump Trucks (to 5/90)
6 ea 10-Wheel Dump Trucks (after 5/90)

This equipment was also effectively used, well maintained, and adequate for the removal of shot rock (photos 49 & 50).

a. Drilling and Blasting. The drilling and blasting contractor for the project was McCaw's Drilling (USA), Inc, working as subcontractor to PCL Civil Constructors, Inc., the prime Contractor for the project. Drilling and blasting started on 14 December 1989, on the left abutment of the auxiliary spillway, and was completed when the subcontractor demobilized from the site

on 26 June 1990. A test panel was not completed. The drilling and blasting subcontractor requested and was allowed to substitute the first presplit blast at the spillway in lieu of a test panel. After some initial problems with hole alignment at the spillway, presplitting progressed satisfactorily. On the right abutment of the RCC Dam, the subcontractor drilled and blasted 210 pre-split holes in one shot. This was done on the assumption that the material was the same as that encountered at the auxiliary spillway and in borings CN-CH-3 and CN-CH-18. The material was actually quite variable, as discussed in Section II, Geology. As a result there was a significant amount of overbreak and irregular presplit lines. Other problems with respect to maintaining well-defined presplit faces are described in Paragraph 5.02a. Drilling and blasting proceeded well ahead of the excavation. Presplitting was generally done in conjunction with production blasting using appropriate delays, stemming, and loading. At the left abutment of the auxiliary spillway and the right abutment foundation trench for the RCC dam section, presplit blasting preceded production blasting. A total of 95 blasts were conducted for the project; 71 during initial blasting, and 24 to complete the modification to the foundation on the left abutment of the RCC dam section described in Paragraphs 2.06a(1) and 5.02a(2). The Contractor's General Blast Plan and select samples of as-built blast reports are contained in Appendix G. These reports provide detailed information on depth, spacing, stemming, delays, and powder factors.

(1) The Contractor maintained two qualified and experienced crews under qualified supervision through January 1990. Thereafter, three crews were on site throughout the drilling and blasting operation, as well as rock bolt installation. In addition, the Contractor was required to maintain all drilling and blasting records, and to deliver complete records to the Contracting Officer within 48 hours for review. Although presplitting was often done on steep slopes and/or in narrow confines, the holes were drilled at the proper angles, depths, and spacing, and were properly loaded and stemmed. Production holes were also properly loaded and stemmed, and only a minimum of radial shatter patterns were observed at the base of some production holes. Records provided by the Contractor were well prepared and contained all of the required information. The Contractor provided the following equipment for drilling for blasting:

2 ea Gardner Denver 3700 Air Track Drills
2 ea Gardner Denver 800 CFM Compressors
1 ea Atlas Copco ROC 722 Hydraulic Track Drill

This equipment was well suited for the drilling required, especially for narrow confines and steep slopes, and was well maintained (photos 51 & 52).

(2) Prior to the start of drilling and blasting operations, the Contractor requested a deviation from the specifications Section 02219, Paragraph 7.10, Item #3 which states "All blasting shall be initiated with an approved electrical system (sequential timer), and controlled by use of MS delays." The deviation proposed the use of a non-electrical initiation system immune to extraneous electricity. The requested deviation was approved by the Contracting Officer on 4 December 1989. The system approved employed a self-

contained plastic tube containing reactive materials that transmit firing signals to various surface and in-hole MS delays. This system can not be initiated by HF radio transmissions or stray electrical energy, flame, friction, or impact. The Manufacturer's literature for this system is included with the Contractor's Blasting Proposals in Appendix G.

(3) After the first presplit blast on the left abutment of the auxiliary spillway, the Contractor requested that the spacing between holes be increased from the specified 24 inches to 30 inches. The Contractor felt that a spacing of 30 inches would yield satisfactory results in the type of rock encountered at the site. The Contractor's proposal was approved on 18 December 1989, and presplit holes were drilled on 30-inch centers for the remainder of the project.

(4) Auxiliary Spillway. The Contractor submitted a blasting proposal for the bench located on the left side of the auxiliary spillway. This proposal is included in Appendix G. Presplit holes were drilled to an average depth of 13 feet and were 2¼-inch-diameter. Stemming consisted of drill cuttings, which were placed above dynamite cartridges. Production holes were drilled to a maximum depth of 16 feet and were 3-inch-diameter. Stemming consisted of drill cuttings, which were placed above ANFO with a 2-inch X 8-inch cartridge of semigelatenous dynamite at the bottom of the hole. Powder factors and the amount of collar stemming varied, dependent on the depth of the hole.

(5) RCC Dam Foundation Trench. The Contractor submitted three separate blasting proposals for RCC dam foundation trench: one for the left abutment and high and low level outlet works; one for the right abutment; and one for the steep-sided center along the channel alignment. These proposals are included in Appendix G. Normal presplit and production blasting was utilized along with cushion blasting in some areas. Details of selected blasts are in the as-built blast reports included in Appendix G. Presplit and production holes were drilled and loaded similarly to those in the auxiliary spillway. Maximum depth of any hole was 20 feet, as specified in the contract. Buffer holes used in cushion blasting were 3-inch-diameter. Stemming consisted of drill cuttings, which were place above semigelatenous dynamite cartridges with a #8 delay (200 msec) at the bottom.

(6) Maintenance Road. The Contractor submitted a blasting proposal for the maintenance road, which is included in Appendix G. Presplit, production, and cushion blasting similar to the blasting at the RCC dam foundation trench was proposed. The maintenance road was deleted prior to completion of drilling and blasting due to difficulties encountered during placement of fill. A haul road previously constructed by the Contractor was substituted.

b. Modification to Left Abutment. On 20 October 1990, all work on the low level outlet works was suspended after some rocks fell out of the left abutment foundation trench upstream of previously installed wire mesh. Continued exposure of the zone of poor quality material described in Paragraph 2.06a(1) resulted in slaking and undercutting of some beds, posing a safety hazard to personnel working below (photo 53). The Contractor was directed to

remove this material by excavating a bench along the left abutment. This zone was determined to be differing site conditions material to be removed by issuing modifications to the contract. These modifications are described in Paragraph 2.06a(1).

(1) Background. As early as March 1990 the Contractor and Resident Engineer expressed concerns about the quality of the rock forming the foundation between the high and low level outlet works on the left abutment. Site visits by the project geologist verified their concerns. The Contractor was requested to submit a proposal to excavate two intermediate benches on the left abutment in the keyway of the foundation trench. The Contractor submitted a proposal at a cost of $59,386.90 to perform the work. The Contracting Officer cancelled the proposed change on 2 May 1990 pending results of the exploratory drilling program described in Paragraphs 3.04a and 3.04b and subparagraphs, which was conducted to determine the extent and nature of this zone of poor quality rock.

Air percussion holes were drilled from the HLOW bench in May 1990. Diamond core borings were originally anticipated, but on 3 May 1990 the Contractor, using the QC subcontractor, submitted a bid in excess of $350.00/LF of drilling and $3,000.00 for each set-up for diamond coring. A scope of work for the drilling was then sent to an A-E geotechnical firm on contract with the Albuquerque District. This firm submitted an acceptable proposal, and drilling was anticipated to start on 15 June 1990. One week before the schedule start, the A-E firm's drill rig was involved in a roll-over accident, further delaying the work. In the meantime, air percussion drilling results, as described in Paragraph 3.05a, indicated that this zone of poor quality material was a stratigraphic feature that did not improve with depth. It was then determined that continued efforts to conduct additional explorations were no longer valid, recognizing that this portion of the foundation trench would require exceptional means to prepare for acceptance as an adequate foundation for the RCC dam.

(2) Contractor's Proposal. On 26 October 1990, the Contractor was requested to submit a proposal to remove "differing site condition" material from the left abutment as Change Item No. 24 (P00024). The Contractor submitted a proposal on 9 November 1990 that included a plan of excavation, a blasting proposal, and a plan to protect the outlet structures. These three features of the proposal are included in Appendix F. The Contractor's total cost of the entire change was estimated at $1,589,954.69.

(i) Excavation. The Contractor's proposed plan consisted of constructing two benches, starting at the top of the slope at the HLOW. The upper bench was to be 15 feet high, and the lower bench was to be 25 feet to 30 feet high. Access was to be via a ramp excavated from downstream of the project. The Contractor proposed to use a backhoe/excavator to cast shot rock to the second bench, then to the LLOW bench, then to the dam foundation floor to be loaded and hauled to the waste area. Mucking of the shot rock at each bench was to follow the drilling and shooting as much as possible.

(ii) Blasting. The Contractor proposed to utilize controlled blasting techniques for this modification designed to limit the maximum particle velocity (peak vector sum [pvs]) at existing structures to 2 inches/second. Presplit and production blasting was proposed using an electric system of initiation, in-hole detonators, and tie-ins. Presplit spacing was designated to be 24 inches.

(iii) Protection of Structures. The Contractor proposed to cover the LLOW conduit with a protective layer of 3/8-inch minus sand. Blasting mats were proposed to eliminate possible damage due to fly rock. Three seismographs were proposed to measure peak particle velocity, amplitude, frequency, and air blast to ensure that the pvs was closely monitored. One seismograph was placed at the HLOW, one at the LLOW, and one within six feet of the presplit line. The calibration certificates for these seismographs are included with the Seismograph Records in Appendix G. After completion of excavation, the contractor proposed to remove the sand protection on the LLOW with a backhoe and hand labor.

(3) Government's Directive. On 26 November 1990, the Contractor was directed to proceed with the work to conform to an excavation plan provided by the Government. Removal of differing site condition material was completed in early February 1991. Final cost of this change was $1,342,238.00. The Government directive is included in Appendix F.

(i) Excavation. The Government directed the contractor to remove the differing site condition material by excavating two benches starting at the HLOW. The first bench would be located at an elevation of approximately 4,655 feet (NGVD), 30 to 35 feet below the HLOW on a bed of competent limestone. The second bench would be approximately 10 feet below that, requiring drilling and blasting of only four feet of limestone. The drill equipment was lowered by crane to the high level outlet works, and the access ramp was built to the intermediate bench. All shot rock was removed directly from the intermediate bench and loaded into trucks with the backhoe/excavator for disposal to a designated waste area. This eliminated the need for multiple handling of the material (photos 54, 55, 56, 57, & 58).

(ii) Blasting. At first, it was felt that only pre-split blasting would be needed, given the highly jointed nature of the rock. The joints, however, tended to attenuate the blast, and individual joint blocks were often too large for the backhoe to handle. After seven blasts, the Contractor was allowed to proceed with production blasting in conjunction with the presplitting. After each blast, the drill rig was lowered onto the HLOW to start drilling another blast series while the previously blasted rock was being mucked out. Use of the seismographs, as proposed by the Contractor, was invaluable in adjusting hole spacing, delay determinations, and powder factors (photos 59 & 60).

(iii) Protection of Structures. The Government generally agreed with the Contractor's proposal, except directed that the vacuum truck be used to remove the sand protection from the LLOW. The vacuum truck was not as efficient as first estimated, and cleanup eventually included the Contractors original

proposal in combination with the Government directive. The protective sand covering the LLOW conduit was effective in preventing damage to the concrete pipe, and the blasting mats minimized the amount of fly rock from each blast (photos 61 & 62). The seismograph located near the presplit line was eliminated when blasting had proceeded to within 75 feet of the HLOW structure. The seismograph located at the LLOW was eliminated after the tenth blast. Prior to that, triggering levels were not attained, or pvs velocities were always less than 0.1 inches/second at the LLOW. In blast number nine, pvs velocity at the HLOW was 8.41 inches/second. The contractor reduced the number of presplit holes per delay and the powder factor. In all previous and subsequent blasts, pvs velocities were less than 2.0 inches/second. Select samples of Seismograph logs are included with the as-built blast reports in Appendix G.

5.06 Foundation Preparation. The Contractor submitted a Foundation Preparation and Joint Treatment Plan on 22 December 1989. Foundation preparation was to consist of:

A. Shaping and filling utilizing mechanical and hand equipment
B. High Volume - Low Pressure Washing
C. Truck-Mounted Vacuum Pick-up System, if needed
D. Waste Disposal.

The Foundation Preparation Plan was amended on 15 January 1990, to reflect that the vacuum system was required by the specifications during all phases of foundation preparation. It was amended again on 31 May 1990 to include high pressure water jetting. The Contractor provided the following equipment for foundation preparation:

1 ea Caterpillar D-9 Bulldozer/Ripper
1 ea Caterpillar D-8 Bulldozer/Ripper
1 ea Caterpillar 245 Trackhoe/Excavator
1 ea IH 4,000 gallon Water Truck
1 ea John Deere 310 Backhoe
1 ea Vac All Vacuum Truck
1 ea Lanada PG4 2500 High Pressure Water Jet
1 ea 190 CFM Air Compressor

a. Auxiliary Spillway. The auxiliary spillway presented some unique problems with respect to foundation preparation. The specifications clearly defined the procedures and equipment that would be required for foundation preparation for RCC placement onto rock, but did not define foundation preparation for RCC placement onto overburden material.

(1) The majority of the auxiliary spillway was excavated into unconsolidated units within the Palomas gravel formation. During excavation, a well consolidated conglomerate was encountered as described in Paragraph 2.06b(1). The chute toe was founded in the conglomerate, which was considered bedrock. The remainder of overlying material was, however, considered common excavation. A large portion of this material formed the bench at elevation

4,705 feet (NGVD) directly below the sill. This bench had RCC placed directly onto it, but was not required to be prepared as rock foundation. The material forming this bench could not be washed with water, but did contain significant amounts of loose material. The same condition existed on the right abutment of the auxiliary spillway where RCC slope protection was required. This material was slightly cemented and could be vacuumed. Vacuuming removed the loose material from the bench without further disturbing the underlying in-situ material (photos 63 & 64). Use of the vacuum truck in addition to hand scaling and cleaning of this bench necessitated it being considered rock foundation contact treatment, which accounts for the majority of the difference in the estimated vs. actual quantities for bid item no. 13 in Table 1-5. Although some suggestions were offered for preparing or protecting the slope on the right abutment of the auxiliary spillway, the Contractor elected to keep this material slightly damp and hand scale and clean the slope as the RCC lifts were placed. This was less than satisfactory, and resulted in some contamination of the RCC along this interface.

(2) On the left abutment of the auxiliary spillway, hand scaling and cleaning, low pressure high volume water washing, high pressure water jetting, and the vacuum truck were all used to prepare the foundation. The vacuum truck proved invaluable, especially in the solutioned limestone zone described in Paragraph 2.04c(2)(i) (photo 65). Where RCC was placed directly on bedrock on the left side of the spillway, the foundation consisted of fault gouge and breccia, with highly jointed argillaceous limestone (photo 66), and required numerous cleanings due to the slaking nature of the material. Bedding mix was placed on the foundation here, as well as in the chute toe prior to RCC placement. Foundation preparation of the left end of the chute toe at the transition from the Palomas gravel to the Madera formation was marginally acceptable, at best. Not all of the loose material could be removed before bedding mix was applied. As a result, the contact between the RCC and bedrock may not be as "tight" as intended in this area of the chute toe. Given the small size of this area, however, this should not effect the design intent of the chute toe.

b. RCC Dam Section. Foundation preparation on the RCC dam section was relatively difficult and required utilization of all the resources specified in the Contractor's Foundation Preparation and Joint Treatment Plan. After initial ripping, drilling and blasting, and shot rock removal, it was obvious that there were zones on both the left and right abutments that would require a substantial amount of work to provide an acceptable foundation for RCC placement. Open, clay-filled joints were cleaned to depths and widths that could be accessed with hand tools and water jetting.

(1) Left Abutment and Bottom.

(i) Hand scaling in conjunction with vacuuming was utilized to remove loose material from the left abutment between STA 1+00C and 1+82C and from the bottom of the foundation trench between STA 3+00C and 3+45C. After removal of loose material, low pressure high volume water washing was required to remove fines and minor clay. Joints in these areas were either tight or slightly open. Where open, joint infilling was removed by hand tools and high pressure

water jetting. After being cleaned, these areas resembled stair steps, as described in Paragraph 2.06a(2)(ii). Both of these areas were mapped, and then covered with dental concrete shortly after final cleanup, resulting in a planer surface for RCC placement (photos 67 & 68).

(ii) From the left side of the HLOW at STA 1+82C and elevation 4,680 to the bottom of the foundation trench at STA 3+00C and elevation 4,578, highly jointed argillaceous limestone, numerous beds of weather sensitive shale and siltstone, a shear/breccia zone, and severely solutioned limestone were encountered as described in Paragraphs 2.04a(3)(iv), 204c(2), and 2.06a(1) and subparagraphs.

(*a*) At the HLOW, the siltstone was removed back to the left side slope by hand scaling and cleaning. The HLOW foundation was then washed with low pressure water and mapped. Slaking material was removed by vacuuming immediately prior to placement of the dental concrete described in Paragraph 2.06a(1)(i)(*a*).

(*b*) The excavated slope between the HLOW and elevation 4,653 feet (NGVD) contained the highly jointed limestone and some shale. This slope was mechanically scaled shortly after excavation and then hand scaled, washed, and vacuumed as the RCC lifts were being placed (photos 69 & 70). This method was marginally satisfactory and required the constant attention of the construction inspectors to verify that the foundation was being properly prepared. Mapping of this slope was completed after preliminary cleanup.

(*c*) The shear/breccia zone and underlying solution cavities were hand cleaned and scaled (photo 71). Following hand scaling the solution cavities were washed with high volume low pressure water, and joint surfaces were water jetted. All remaining loose material and wash water was removed by vacuuming. Mapping of this area was completed shortly after final cleanup. This zone was then filled with dental concrete to an elevation of 4,648 feet (NGVD) (photo 72).

(*d*) Between elevation 4,658 feet (NGVD) and the LLOW, the slope consisted primarily of joint faces with abundant open, clay-filled joints. The slope between the LLOW and the bottom of the trench consisted primarily of joint faces and well defined presplit faces with some open joints and abundant fractures. Final cleanup of these slopes consisted of hand cleaning, water washing, and vacuuming and was performed as the RCC was being placed (photo 73). On the lower slope, there were numerous broken blocks of rock which were slightly loose, but could only be removed with significant effort using hand tools. Some of these blocks were left in place. The resulting slope was highly irregular, making placement of dental concrete difficult (photo 74). Again, it was also difficult to constantly inspect and verify the adequacy of foundation preparation on these slopes. Both slopes were mapped after preliminary cleanup was completed.

(*e*) The LLOW bench was similar to the bottom of the trench. After hand scaling and cleaning, the bench surface was highly irregular with abundant open joints. Joints were cleaned by hand and water jetted. All remaining

loose material and wash water was removed by vacuuming immediately prior to placement of dental concrete (photo 75). Mapping of the LLOW bench proceeded in conjunction with the cleanup and was completed immediately prior to dental concrete placement.

(2) Right Abutment. Hand scaling in conjunction with vacuuming was utilized to remove loose material from the right abutment between STA 4+55C and STA 6+36C. After removal of loose material, low pressure high volume water washing was required to remove fines and minor clay. Joints between STA 5+70C and STA 6+36C were either tight or slightly open. Where open, joint infilling was removed by hand tools and high pressure water jetting. After being cleaned, this area resembled stair steps, as described in Paragraph 2.06a(2)(ii). Between STA 4+55C and STA 5+70C, there were abundant open, clay-filled joints up to two feet wide and an area of severe alteration as described in 2.04c(2)(ii). Removal of joint infilling and altered material required the use of a backhoe and hand labor and water jetting, followed by high volume low pressure water washing and vacuuming (photos 76 & 77). Rock bolts were used to anchor the more competent blocks of limestone to prevent them from slipping along bedding planes during mechanical cleanup. The foundation trench between STA 4+55C and STA 6+36C was mapped, and then covered with dental concrete shortly after final cleanup, resulting in a planer surface for RCC placement.

(i) The remainder of the right abutment foundation trench between STA 3+45C and STA 4+55C was relatively steep and was hand scaled and washed with high volume low pressure water prior to RCC placement. This portion of the foundation was mapped well ahead of RCC placement. Final cleanup consisted of hand cleaning, water jetting, and water washing in conjunction with vacuuming, and was performed during RCC placement. Once again, this made it very difficult for the inspectors to verify that the foundation had been properly cleaned and prepared.

c. Protection of Weather-Sensitive Materials. The contract specifications did not provide for a means of protecting weather sensitive materials. A number of shale and siltstone units were exposed during excavation of the left abutment of the RCC dam foundation trench and the left abutment of the auxiliary spillway. These units are discussed in Paragraph 2.06 and are described in detail in Table C-1 in Appendix C. In addition, a significant amount of altered limestone was exposed on the right abutment of the RCC dam foundation trench. All of these units slaked readily shortly after exposure to the air. Much of the foundation for the auxiliary spillway was in unconsolidated Palomas gravel, which was also extremely weather sensitive.

(1) In November 1990, the Contractor was requested to submit a proposal to apply shotcrete to the weather sensitive materials exposed in the foundation trench and was provided shotcrete specifications. The Contractor's estimate for applying 2,500 square yards of shotcrete was $530,289.86. This was a non-negotiable amount that the Government considered excessive, and the request for proposal was cancelled.

(2) In January 1991, the Contractor was requested to submit a proposal to apply a polymer soil stabilizer on 700 square yards of the left abutment of the RCC dam foundation trench between the HLOW and the LLOW. The Contractor's estimate for this work was $16,453.17. At that time, it was obvious the previous changes and modifications had increased the cost of the contract to well beyond the award cost, and there were concerns that funding would not be available for completion of the project and the mandated post completion reports and inspections. The request for proposal was, therefore, cancelled. Much of the weather sensitive material was eventually covered during placement. Those areas still exposed are discussed in Section VII, Lessons Learned and Possible Future Problems.

d. Safety Precautions Against Slides and Rockfalls. Hand and mechanical scaling of loose material was the primary means by which the contractor controlled potential hazards due to rock falls and/or slides. More often than not, the Contractor had to be directed to perform scaling when construction inspectors noted unsafe working conditions related to rock falls.

(1) Wire Mesh. The Contractor submitted a Wire Mesh Installation Plan in December 1989 which is included in Appendix F. The zone of sheared, jointed, and brecciated rock encountered on the left abutment of the RCC dam foundation trench and described in Paragraph 2.06a(1) presented a rock fall hazard to personnel working below on the LLOW and in the bottom of the foundation trench. This material deteriorated over time, eventually resulting in a suspension of work and a modification to the contract to construct an intermediate bench as described in Paragraph 5.05b. In May 1990, the Contractor was directed to install wire mesh on the left abutment between the HLOW bench and the LLOW bench. The wire mesh extended from STA 3+00U to STA 4+00U. It was anchored on the outside edge of the HLOW bench with rock bolts on six-foot centers, draped over the side and anchored to the base of the slope on the left side of the LLOW with rock bolts on six-foot centers at elevation 4,620 feet (photos 78 & 79). The wire mesh was not installed until August 1990 and was removed in November of 1990 when excavation of the intermediate bench started. The rock fall which occurred in October of 1990, prompting the suspension of work, was within the key way portion of the foundation trench, upstream of the wire mesh.

5.07 Foundation Rock Bolts. The Contractor submitted a Rock Bolt Installation Plan in December 1989 that contained specifications for the chemical grout, rock bolt, and the tension jack and is included in Appendix F. Foundation rock bolts were installed on the right abutment of the RCC dam and the left side of the HLOW on the left abutment of the RCC dam. An unsuccessful attempt was also made to install rock bolts at one location on the left abutment of the auxiliary spillway. There were some problems encountered with rock bolt installation that required modification of the installation and pull out test procedures. Rock bolt locations are illustrated on the foundation maps in Appendix D.

a. Auxiliary Spillway. Two rock bolt locations were marked on the left abutment of the auxiliary spillway where a large joint block of limestone was being undermined by the slaking of an underlying shale bed. While drilling

for the rock bolts, a partially filled clay cavity or open joint was encountered five feet back from the abutment face. One cartridge of resin disappeared into this void and drill cuttings were exiting joints in the abutment instead of the drill hole. Thereafter, no effort was made to install rock bolts at this location. It was recommended that the large block of limestone be removed, but it was still in place when the project was completed.

b. Right Abutment of RCC Dam. On the right abutment of the RCC dam between STA 4+55C and STA 5+70C, rock bolts were utilized to anchor large joint blocks of competent limestone to prevent slippage while open joints and alteration zones were excavated with a backhoe as described in Paragraph 5.06b(2). Eighteen 15-foot-long rock bolts were installed in this area in May 1990 (photo 80). Downstream of the RCC structure a number of large joint blocks were observed with potential slippage planes dipping toward the channel bottom. The concern was that these blocks, if they slipped, would alter the geometry of the canyon and interfere with flood flows. Although no slippage of these blocks had been noted to date, the Contractor was directed to install a line of 15-foot-long rock bolts on 10-foot centers parallel to the presplit line at elevation 4,660 feet between STA 2+25D and STA 3+85D. A total of 17 rock bolts were installed in January 1991 (photo 81).

c. HLOW. Rock bolts were used to anchor joint blocks on the left side of the HLOW to hold these blocks in place until work on the box culvert and U-channel were completed. In May and June 1990, 24 rock bolts ranging in length from 10 feet to 15 feet were installed (photos 82 & 83).

d. Installation and Testing. Of the first 26 rock bolts installed on the right abutment of the RCC dam and the HLOW, 23 failed during pull-out tests. Three bolts on the HLOW did not fail and were locked off at 40,000 kPa. The Contractor's submittal indicated that a 1 inch-diameter bolt would be used in conjunction with a 1-5/8-inch-diameter hole as specified by the supplier, Williams Form Engineering, Inc. The supplier also recommended a 3-foot bonded length using three cartridges. After installing 18 unsuccessful bolts on the right abutment, the Contractor felt that the holes were becoming contaminated with clay. After consulting the supplier, it was determined that a good mix of the two-part resin was not being achieved. The Contractor was told to cut the ends of the bolts at a 45° to facilitate a better mix. Eight holes were then drilled at the HLOW, and seven bolts were installed using five cartridges each. Six bolts failed the pullout test. The driller suggested that the spin time and speed were excessive. The supplier was notified again and agreed that the spin time should be decreased to 20 seconds, and spin speed should be decreased to about 250 rpm. At that time, it was pointed out that the hole diameter was two inches using a bit 1-7/8 inches in diameter. The supplier said that this was far too large a diameter hole for a 1 inch diameter rock bolt, and that additional tubes would not solve the problem. The Contractor still wanted to try a lower spin speed and time. Another rock bolt was installed in the remaining open hole using a spin speed of 250 rpm for 15 seconds. This bolt also failed the pull out test.

(1) The Contractor asked if the old holes where the bolts failed could be reamed out, and a larger rock bolt installed. It was agreed that either a smaller diameter hole or a larger diameter rock bolt would be the only solutions. The Contractor then pulled out a failed rock bolt from the HLOW. During reaming, the drill bit broke the cartridges and mixed the chemical grout which was still in the hole. The quick-set grout worked as designed, and the drill bit and 10 feet of steel remain in that hole. The Contractor then filed a bit down to 1-1/2-inch-diameter and drilled and installed another 15-foot-long rock bolt. The unbonded length was drilled as before, and the bonded length was drilled with the smaller diameter bit. This bolt passed the pull out test and was locked off at 39,500 kPa. The Contractor elected to use four cartridges for all successive rock bolts. Of the 11 additional rock bolts installed on the HLOW, seven passed the pull out test (photos 84 & 85).

(2) The Contractor requested permission to tremmie pump cement grout into all of the failed bolts and retest them as cement-grouted bolts. The Contractor was given approval of that plan and informed that the bolts would be accepted if they passed a revised pull out test. Most of these rock bolts were located along the right abutment and were intended to prevent large, competent blocks of limestone from slipping along bedding plane joints prior to RCC placement. It was determined, therefore, that a pull out test of 24,000 kPa would be sufficient. An additional five feet of bonded length was specified for the 15-foot-long rock bolts. All previously failed rock bolts that were rebonded with cement grout easily met the 2',000 kPa requirement with no signs of strain or pull-out.

(3) The 17 rock bolts described in Paragraph 5.07b(1) were also installed using cement grout. These bolts were pull tested and locked off at 35,000 kPa to 37,500 kPa.

VI - FOUNDATION CHARACTER, TREATMENT, AND INSTRUMENTATION

6.01 Character of Foundation - General: The character of the exposed foundation was highly variable. The foundation surface varied from highly irregular in parts of the RCC dam foundation trench to well formed and planar at the auxiliary spillway and those areas where presplit blasting worked well. The variation was the result of geologic conditions encountered during excavation.

a. Foundation Surface.

(1) RCC Dam Section. The foundation surface for the RCC dam section was composed primarily of bedrock of the Madera formation, which consisted of limestone and minor shale with zones of altered and/or sheared and brecciated material. Some consolidated Mud Mountain fanglomerate formed the foundation on the left side of the dam between STA 0+00C and STA 0+90C. These units are described in detail in Paragraph 2.04b(1) and subparagraphs, as well as in Table C-1 in Appendix C.

(i) Irregularities in the foundation surface were generally the result of unanticipated geologic conditions. Breaking of the bedrock along joints resulted in poor presplit faces as described in Paragraph 5.02a and in stair-stepping as described in Paragraph 2.06a(2)(ii). The most severe irregularities were the result of the removal of weather sensitive shale and siltstone, altered limestone, and open joint and cavity infilling material as described in Paragraphs 2.06a(1) and subparagraphs, 2.06a(2), and 2.06b(2). The majority of these areas required a significant amount of dental concrete to provide a surface on which mechanical rollers could be used for the placement of RCC.

(2) Auxiliary Spillway and Inspection Trench.

(i) The left side of the auxiliary spillway between STA 1+40S and STA 2+10± was founded on bedrock of the Madera formation. Much of the bedrock in this area was highly jointed and contained a number of shale beds and a large area of severe solutioning as described in Paragraph 2.04c(2)(i). The resulting extremely irregular surface required a significant amount of dental concrete as described in Paragraph 2.06b(2) (photo 39).

(ii) The remainder of the spillway, as well as all of the earth embankment section inspection trench were founded in unconsolidated to well consolidated Palomas gravel. Although bedding was evident, there were no joints observed in the formation. Excavation was relatively easy using mechanical scrapers. Cut slopes were well maintained and did not slough. This resulted in planar surfaces ideal for RCC placement (photo 86).

b. Engineering Characteristics of Soil and Rock. The engineering characteristics of the soil and rock are discussed Paragraphs 3.03i, 3.03j, 3.03k, and 3.05b.

6.02 Foundation Treatment. Dental concrete and bedding mix, which were specified in the contract, were the only materials used for foundation treatment. As discussed in Paragraphs 5.06c(1) and 5.06c(2), other methods of foundation treatment were not included in the Contract Specifications, and efforts to modify the contract to include them were unsuccessful.

a. Dental Concrete. Dental concrete, a 2500 psi, 3/4-inch aggregate mix, was used extensively at this project in the RCC dam section foundation trench and on the foundation for the left side of the auxiliary spillway. The final surface geometry of the foundation was highly irregular and not conducive to roller placement. Dental concrete was used to provide an acceptable foundation for placement. Depending on the volume of dental concrete for any given area, as well as access to the area of placement, different methods were utilized for placement. Much of the foundation trench was completely covered with dental concrete far in advance of RCC placement. A total of 4,674 cubic yards of dental concrete was required for this project.

(1) On the left side of the auxiliary spillway, 1,000+ cubic yards of dental concrete was required to fill in irregularities and the large solution cavities described in Paragraph 2.04c(2)(i) (photos 40 & 41). Most of this concrete was placed directly from the mixer truck and vibrated into place.

(2) On the right abutment of the RCC dam section foundation trench, 800+ cubic yards of dental concrete was placed into the voids and irregularities described in Paragraphs 2.06a(2)(i) and 2.06a(2)(ii) using a crane and a 1.5-cubic yard concrete bucket. The concrete was vibrated into place using hand held pneumatic vibrators.

(3) On the left abutment, the bucket and crane were also used to place 900+ cubic yards of dental concrete onto the irregular "stairstep" surface described in Paragraph 2.06a(2)(i), the high level outlet works bench, and the low level outlet works bench (photo 87).

(4) A pumper truck was utilized to place nearly 1,700 cubic yards of dental concrete into the solution cavities on the left abutment of the RCC dam section foundation trench described in Paragraph 2.06a(1) and the bottom of the foundation trench (photo 88).

(5) The remaining dental concrete was placed by hand using the bucket and crane as the RCC lifts were placed. The concrete was placed into voids and irregularities not accessible to the rollers and vibrated into place. The dental concrete was usually placed after every two lifts of RCC were placed.

(6) All large areas of dental concrete were cured for 14 days by wetting, covering the exposed surface with saturated burlap cloth, and then covering that with weighted plastic sheeting (photo 89). The water truck was used to periodically spray those areas that appeared to be drying. Prior to RCC placement, these surfaces were water jetted to remove any rind on exposed aggregate. Water jetting did not work well because it only removed a portion of the rind. The Contractor, however, was not directed to utilize wet sandblasting or other methods.

b. Bedding Mix. A neat cement bedding mix was used to fill in those joints and irregularities into which dental concrete could not be used due to the 3/4-inch aggregate. A thin veneer of this bedding mix was applied by hand to nearly all the foundation immediately prior to RCC placement, including those areas where the entire foundation was covered with dental concrete. Bedding mix was delivered utilizing the crane and concrete bucket.

(1) An effort was made to keep foundation surfaces moist prior to the application of bedding mix. The need to keep each lift of RCC moist, in addition to the foundation, strained the Contractor's system of moist curing. There were two or three laborers responsible for misting the foundation and RCC surfaces using hoses and nozzles. They were also often called upon to assist in other work, making it difficult for the construction inspectors to ensure that proper wetting was being performed (photo 90).

6.03 Foundation Approval and Mapping.

a. For those portions of the foundation requiring large quantities of dental concrete, the foundation was inspected by the Project Geologist after final cleanup and mapping. If acceptable, the Resident Engineer was provided a map outlining those areas of the foundation deemed acceptable for dental placement. The Resident Engineer would then approve the foundation, having elected to assume that responsibility. These areas are discussed in Paragraphs 6.02a(1) through 6.02a(4), and the foundation approval map is included as Plate No. A-13 in Appendix A. The remainder of the foundation, including the dental concrete surfaces, was approved on a daily basis during RCC placement by the construction inspectors and/or the Resident Engineer. The foundation maps are included in Appendix D, and detailed lithologic descriptions of the rocks encountered are included in Appendix C.

b. The foundation mapping was performed by the Project Geologist with assistance from other Geotechnical Branch personnel. The entire RCC dam section foundation trench was mapped. Upstream and downstream sidewalls of the trench which were not used as forming for RCC or conventional concrete were not mapped. The inspection trench for the earth embankment section was not mapped. It was excavated into a nearly homogeneous, horizontally bedded, slightly consolidated, sandy clayey gravel of the Palomas gravel formation. At the auxiliary spillway, the Madera limestone on the left side and left abutment was mapped, as well as the chute toe trench and the 1V on 1H slope between elevation 4,715 feet (NGVD) and 4,705 feet (NGVD) along the sill axis. The Palomas gravel exposed in the remainder of the spillway between the chute toe and sill could not be mapped. The units mapped at the chute toe and the sill are easy, however, to correlate across the remainder of the spillway slope.

(1) Due to the concerns with respect to the quality of the foundation and the potential for seepage, mapping was performed using a scale of one inch equals 5 feet (1:60). This was done in order to provide a maximum amount of detail to illustrate the geologic features of the trench. Mapping progressed in sections as foundation cleanup was finished. For each section, a 5-foot grid was laid out to assist in accurately depicting all features. For those

sections with highly irregular surfaces, all features were projected normal to an "ideal" plane that matched the overall slope as nearly as possible. This removed the distortion that would otherwise have resulted by projection to vertical or horizontal planes (plan view).

(2) In the areas described in Paragraphs 6.02a(1) through 6.01a(4), mapping could not be completed until final cleanup, which was conducted immediately prior to dental concrete placement. Often, placement of dental concrete on portions of the trench that had just been mapped and approved was occurring within 10 feet of ongoing mapping. This was especially critical in those areas of the foundation consisting of weather sensitive material.

6.04 Foundation Instrumentation. At completion of the project, there was no instrumentation on site. The types and quantities of potential instrumentation was a controversial subject discussed and argued in numerous pre-design meetings. It was determined that, given the short duration of the design flood and the likelihood of not having personnel from the local sponsor available to read the instruments, instrumentation of any type would not be required. The same reasoning was used to determine that foundation grouting would also not be necessary. In addition, it is likely that the majority of horizontal movement of water through the abutments would be along the vertical and near vertical fractures. It would be difficult, therefore, to intersect enough fractures with a piezometer to provide meaningful data. There are some settlement/alignment caps on the upstream railing at the top of the dam.

Photos 91, 92, and 93 are views of the project upon completion.

VII - LESSONS LEARNED & POSSIBLE FUTURE PROBLEMS

7.01 Lessons Learned - General. A number of problems were encountered during construction of this project. Although one root cause for any given problem may not be recognizable, there are certainly some identifiable sources that were preventible. As a result of the problems encountered, modifications and variations in estimated quantities added nearly 2.8 million dollars to the cost of the contract. Details of most of these modifications are discussed in Section 2.06. The intent of this section is to point out some of the mistakes made during all phases of the project and offer suggestions for avoiding the same mistakes in the future.

a. Preconstruction Investigations.

(1) The amount, types, and quality of the preconstruction investigations were insufficient. Cuchillo Negro Dam is a dry dam for flood control only. Compared with many other Corps of Engineers civil works structures, is relatively small. Without close examination, the site geology appears to be simple and straightforward from both a lithologic and structural standpoint. As it turned out, the geology, especially the structural features, was more complex than anticipated. There were not enough subsurface explorations to adequately describe the geologic conditions at this site, especially for the RCC dam section.

(i) Although the general site had been selected for the location of Cuchillo Negro Dam, the final alignment was not determined until early 1989. One alignment was explored with three diamond core borings in 1984. The alignment was then moved 200 feet upstream and explored with two diamond core borings in 1988. The final alignment fell midway between the two explored alignments. One diamond core borehole was drilled at the stilling basin in 1988, and another was drilled at the inlet tower for the LLOW in 1989. Also in 1989, two 4-inch-diameter borings were drilled near previous borings on the far left and far right abutments for the purpose of conducting a borehole camera survey. Individual borings are discussed in detail in Paragraph 3.01. As a result, there were no borings located on the actual dam alignment. Even when existing explorations were projected to a cross section along the dam axis, much of the alignment was not explored. The far left abutment, the far right abutment, and the channel bottom were the only portions explored. A 110-foot-thick section of the stratigraphic column was not encountered by any of the borings. As discussed in Paragraphs 2.06a(1) and 2.06a(2), some weak shale and siltstone, some highly jointed limestone, the shear/breccia zone, and the severely altered limestone were all located in t his 110-foot interval. Additional borings on each abutment, along the alignment of the HLOW, and along the alignment of the LLOW would likely have encountered some of these features. However, given the narrow, near vertical dimensions of the most severely solutioned limestone zones, some features may have remained undetected.

(ii) All borings drilled at the site were vertical. Given that bedding planes were dipping at an angle of 25°±, some borings should have been drilled at an angle normal to the bedding planes in order to determine the actual thickness of individual rock units. Vertical borings tend to deviate in dipping strata, especially if rocks with variable hardness and density are encountered. A regional joint survey was not performed, but it was noted that a majority of the joints were near vertical or steeply dipping. Borings drilled in different directions at angles other than vertical would have better defined the joints existing at the site.

(iii) Due to budget constraints, accurate surveys were not performed for most of the boring locations, and none were performed for the as-drilled sites. As a result, the degree of accuracy was less than adequate. Some borings were located in the field as much as 50 feet from where shown on the plans. Accurate locations and elevations are always important, but become critical when borings in dipping strata are projected to cross sections that are used to determine the subsurface conditions.

(iv) Some of the core from the diamond core borings was not logged, particularly when the boring's purpose was to confirm what had previously been determined in adjacent borings. A few of the other explorations were not logged, and some logging was inadequate and/or incomplete. Seven or more individuals were responsible for logging the core during the multiple phases of exploration. Ideally, one qualified geologist should relog all core in order to provide consistent lithologic descriptions of identical units encountered in separate borings. None of the core for this project was relogged. In addition, relogging the core at a later date may identify zones of weather-sensitive material or other features not identified during the often hectic field logging.

(2) Recommendations. The importance of a well designed exploration program cannot be overemphasized. The program must be adequate to eliminate as many foundation uncertainties as possible and be properly executed, regardless of the size or function of the project being designed. Without a proper exploration program, major cost overruns due to changed conditions are almost guaranteed. Murphy's Law operates just has often in geology as it does in other disciplines. Most of the geologic problems encountered during construction of Cuchillo Negro Dam were unanticipated only because they were unexplored, ie. you can't anticipate what you don't know. Early on in the planning process, sufficient funds and time should be set aside for an adequate investigation program. The following is a list of recommendations which, in most cases, should be standard operating procedures, but are emphasized to prevent similar problems on future projects:

(i) Design and implement an exploration program that provides an adequate number of subsurface explorations for all features of a project, even if access, cost, or time is a problem.

(ii) Conduct detailed preliminary field investigations to include such items as a joint survey, and look at the perimeters of a site to observe any surface structural features that may extend into the project. If possible,

conduct a lineament survey to detect and investigate linear features that may be faults, especially in rift or other documented fault zones. It is easy in hind sight to find these features after excavation has started, but it may also be possible to observe and verify them ahead of design.

(iii) All explorations should be properly located and properly logged by qualified personnel. If possible, a geologist should relog all core, paying particular attention to any features which could present a problem during or after construction. Relogging should be done before the core is damaged by continued exposure to the weather and numerous moves and as far ahead of final design and/or construction as possible.

(iv) Survey locations of all exploration sites and verify that topographic maps being used as base maps for design are accurate. It appears that the topographic map used for this project had a slight horizontal offset error. The result was that final excavated slopes were not at the depth below original ground surface that was indicated on the plans.

b. Foundation Preparation and Treatment.

(1) The plans and specifications for this project were well written and thorough, but a few additions would have prevented some of the problems encountered during construction. These additions would not have likely been anticipated when the plans and specifications were written and are mentioned only to prevent a similar situation from occurring in the future.

(2) Recommendations.

(i) Protection of Weather Sensitive Materials. The contract should include a section and bid item for the protection of weather sensitive materials. This is very important when foundation bedrock is sedimentary rock, but should be included in all plans and specifications unless there is overwhelming evidence that there are no weather sensitive materials present. Shotcrete (standard or reinforced) and polymer binder could have proven very useful for this project. By the time their use was anticipated, there were already significant cost overruns, and the requests for proposals for these items as described in Paragraphs 5.06c(1) and 5.06c(2) were cancelled. Split bidding could be utilized in order to reduce the variations in estimated quantities that would result if these items were specified, but not utilized.

(ii) Foundation Cleaning. The contract specified the equipment and methods the Contractor would utilize to perform foundation cleaning. These specifications were adequate in that respect. There was no way, however, to direct the sequence or timing of foundation cleanup. After most loose material which could be mechanically mucked out or hand excavated was removed from the foundation trench, cleanup was not initiated until shortly before scheduled dental concrete or RCC placement. In some cases, even a low pressure high volume water wash was not done, and portions of the foundation trench remained uncleaned for 8 or more months.

(*a*) Mapping of the foundation required it to be clean in order to observe the features to be mapped. As a result, mapping was done in "spurts" under pressure to complete it in time for scheduled placements. In addition, it was difficult to determine where to place rock bolts. By the time some rock bolt locations could be identified and marked, the drill rig had advanced to well beyond that location. The effort which was required to move back onto these sites resulted in a claim. While it may not be in the best interest of the Government to actually direct the Contractor's sequencing of foundation cleanup, some control is needed. A solution, therefore, would be to have a separate "preliminary cleanup" bid item. This would allow the Project Geologist a reasonable amount of time to map the foundation, mark rock bold locations, delineate those areas where further cleanup and dental concrete may be needed, and designate areas requiring protection from the weather.

(*b*) The Contractor elected to do the final cleanup of much of the foundation during RCC placement. Given the steepness of the slope, and the effectiveness of debris removal with the vacuum truck, this was a viable option but required close monitoring. In order to prevent any problems similar to those discussed in Paragraph 5.06b and subparagraphs, possible solutions include: 1) There should be a clause specifying that final foundation cleanup must be completed and the entire foundation approved before any placement is allowed to commence, or, 2) If the Contractor intends to perform final cleanup and preparation of the foundation contemporaneous with placement, there should be a clause stating that a crew shall be designated for performing only that work, and that cleanup and preparation shall proceed at least 24 hours ahead of placement.

c. Excavation Procedures.

(1) The Contractor's Excavation Plan of November 1989 is included in Appendix F. The Contractor elected to excavate and construct the auxiliary spillway first. The drilling and blasting proceeded to the RCC dam section foundation trench after drilling was completed on the left side of the auxiliary spillway. When the spillway chute toe elevation was changed after encountering consolidated conglomerate, the drilling operation was moved back to the spillway to drill and blast the chute toe. When drilling and blasting was completed in June 1990, the drilling subcontractor demobilized from the site. There was no qualified crew remaining to perform any additional drilling and blasting or rock bolt installation. One of the costs for performing the modification to the left abutment described in Paragraph 5.05b and subparagraphs was to remobilize the drilling subcontractor to the site. In addition, the foundation trench was open for as much as one year before placement began. This contributed substantially to the deterioration of the weather sensitive materials.

(i) In addition to the Excavation Plan, a number of other important plans were submitted by the Contractor, including the General Blast Plan, Foundation Preparation Plan, and the RCC Placement Plan. None of these plans were submitted by the field office to geotechnical or other design personnel for review.

(ii) When the consolidated conglomerate described in Paragraph 2.06b(1) was encountered, the Contractor was instructed to relocate the chute toe from the design elevation of 4,628 feet (NGVD) to an elevation of 4,687 feet (NGVD). The specifications state that the Government reserves the right to found the chute toe on the conglomerate if it proves to be adequate. The Contractor's Excavation Plan, however, assumed that the excavation would continue to the design elevation. The Contractor was able to claim on the basis of a change to the contract arguing that this was a major design change which resulted in idle equipment time and the rehandling of fill material that had been wasted. The Government had reviewed and approved the Excavation Plan. The Contractor was awarded a modification in the amount of $80,000 and received additional compensation through the variations in estimated quantities clause.

(2) Recommendations.

(i) All plans submitted by the Contractor should be reviewed and discussed by design, geotechnical, and construction personnel prior to COE approval. If only one or two personnel are reviewing and approving the plans, there is a chance that a critical issue will be overlooked. If these plans can not be made part of the initial bid package, then a clause should be included in the contract stating that they must meet COE approval before a Notice to Proceed can be issued.

(ii) Even in the best case scenario, there is the potential of unanticipated geologic conditions which may have a significant impact on a project's cost and schedule. One possible solution, which has been done by other Government agencies, is to issue a separate contract for the excavation portion of the project. This contract should include excavation, primary cleanup, protection of weather sensitive materials, and all permanent treatment, such as dental concrete. If major problems are encountered, plans and costs for altering the foundation can be carefully determined, or the designers will have time to change the design, or even move the alignment.

(iii) Additional drilling and blasting, as well as rock bolt installation, could be required at any time up to completion of the project. The contract should specify that this capability should be available for the entire length of the construction period and that these services may be required in any area of the project, even if it is well after previous drilling and blasting. Often, it is only after significant cleaning of the foundation has been completed before the need for rock bolts is identified. In the case of the rock bolts installed on the right abutment downstream of the RCC dam described in Paragraph 5.07b, it wasn't until excavation had proceeded well below this area that the need for rock bolts was recognized. The drill rig had to set up in this area a second time, long after performing presplit drilling and blasting. The Contractor was successful in claiming additional compensation via Modification No. P00043 for $5,400.

d. Personnel. During construction of this project, there were concerns raised regarding the number and qualifications of Corps personnel assigned to the project. When projects of this magnitude are constructed, personnel

representing different areas of expertise are assigned to the project and provided with defined responsibilities. Ideally, the Resident Engineer can utilize these personnel and either delegate decisions to them, or make decisions based on their recommendations. Either option is acceptable, as long as it is defined and agreed upon prior to the start of construction.

For this project, the Resident Engineer assumed the responsibilities of foundation approval and the approval of individual blast reports. The project geologist also provided limited foundation approval. This led to some confusion on the part of the Contractor. A case in point occurred on 20 September 1990. The Contractor was ready to place dental concrete on the foundation of the LLOW. Although the Project Geologist had approved the foundation, the Contractor was reluctant to place the concrete without the approval of the Resident Engineer, who was off the site for the day. Placement was authorized only after telephone approval was given by Construction Branch at the District office in Albuquerque. The Resident Engineer returned to the site as placement was beginning. An overnight storm flooded the project the following day, covering the LLOW. Had the dental concrete not been placed, the LLOW foundation would have required recleaning, which had already occurred once and resulted in a claim and modification for $71,696. Incidents such as this can lead to confusion, friction, and misunderstanding between the Government and Contractor personnel. All other foundation related requests for information, such as rock bolt location and installation determinations, slope protection measures, and delineation of areas requiring dental concrete and/or additional scaling were referred to the Project Geologist for recommendations.

(1) Project Geologist. A full-time, on-site Project Geologist was not assigned to this project. Had there been no major unanticipated geologic conditions, and had the Contractor's excavation and foundation preparation procedures been better controlled, then a full-time, on-site geologist may have been able to recognize potential foundation problems and recommend solutions before they impacted construction progress. The unanticipated geologic conditions which were encountered during excavation of the foundation trench were recognized as potential problems at an early stage by both the Government and the Contractor. Efforts to investigate the foundation and recommend remediation procedures were delayed due to logistical and procedural problems as discussed in Paragraph 3.04a(1). Meanwhile, excavation of the foundation trench continued, and until it was nearly complete, the nature and extent of the foundation problems could not be fully understood. The Project Geologist visited the site on a regular basis and when requested, as did other Engineering and Planning Division personnel. All planned foundation mapping was completed without delaying the work in progress, and technical advice was provided when requested.

(2) Recommendations. When staffing for a project is being determined, careful consideration of the number and qualifications of personnel is critical. Once the staffing is determined, roles and responsibilities should be well defined and then executed by those they are delegated to. A meeting for the purpose of defining the duties and responsibilities of all personnel involved with the project should be held between construction, design,

geotechnical, and Contractor personnel prior to the start of work.

7.02 Possible Future Problems.

a. Impoundment of Water. Shortly after the project was completed in July - August 1991, significant rainfall occurred in the watershed of Cuchillo Negro Creek. Debris carried by the flood waters blocked the ports on the intake tower, causing water to back up behind the dam. The resulting pool reached a high water elevation of 4,650 feet (NGVD), 32 feet above the channel bottom. During an inspection in October 1991, the pool elevation was at 4,634 feet (NGVD) and three ports above the waterline were still clogged with debris from the original flood. Water was flowing through the outlet at less than 20 gallons per minute, and it was estimated that between 50 and 60 acre-feet of water were present in the reservoir.

(1) Three of the four primary joint sets described in Paragraph 2.04a(1) contain joints that extend through the entire width of the dam section. Most of these joints contain a clay infilling. One of the primary reasons abutment leakage through these joints was not a major concern was that Cuchillo Negro was designed as a dry dam. Short term wetting of the joint infilling material is not anticipated to be a problem. Under design conditions, the erosion of the joint infilling material should be minimal, with no significant seepage anticipated during the life of the project. Although no seepage through the abutments has been observed to date, continuous wetting by stored water over a long period of time could lead to accelerated erosion of the infilling material with subsequent seepage downstream.

(2) Other than the chute toe and left side, the majority of the auxiliary spillway is founded primarily on semi-consolidated and unconsolidated units within the Palomas gravel formation. Continued pooling of water could saturate this material and weaken the foundation below the auxiliary spillway. This, in turn, could lead to some settlement and cracking of the auxiliary spillway sill.

(3) Recommendations. Continuous pooling of water behind the dam must be prevented. To ensure that the problems described in Paragraphs 7.02a(1) and 7.02a(2) do not develop, the project must be operated as designed. The Albuquerque District has designed a trashrack to prevent debris from entering and clogging the ports on the intake tower. Once the system of debris catchment is installed, it should be cleaned on a regular basis by the operator. This will allow for proper drainage of the reservoir and restore the project to design intent.

b. High Level Outlet Works. Under design conditions, the HLOW will eventually experience flows. The present HLOW structure diverts these flows back into the channel directly above the outlet of the LLOW.

(1) With continued flows through the HLOW, water cascading down the sides of the left abutment will erode the weaker shale, siltstone, and the solutioned limestone exposed on the left abutment downstream of the face of the dam. This problem was recognized during construction, but efforts to

resolve it at that time were unsuccessful, as described in Paragraphs 5.06c(1) and 5.06c(2). Without protection, erosion may proceed into the abutment, as well as upstream toward the downstream face of the dam, and eventually undercut the more competent beds of limestone and the HLOW "U"-shaped channel. Falling blocks of limestone could block the outlet for the LLOW.

(2) Recommendations. Erosion due to HLOW flows will be easily detected during periodic inspections. If it appears that the outlet works structures could become threatened, then remedial measures may be required. Slope protection, such as shotcrete could be applied to the slope between the HLOW and the LLOW, or the outlet channel for the HLOW could be altered and extended to divert water back into the channel well downstream of the dam. Blockage of the outlet for the LLOW would not be critical after siltation has reached the top of the intake tower. Prior to that time, however, any debris could be easily removed.

c. Weather Sensitive Material. There is still a significant amount of weather sensitive material exposed at the site, especially on the left abutment of the RCC dam and the left side of the auxiliary spillway.

(1) Continued exposure to the atmosphere will result in additional slaking of this material. Observing the existing natural angle of repose in the canyon walls, it appears that the slaking will not proceed to a point where the major structures are threatened. At the left abutment of the dam, however, this slaking may eventually result in the undermining of the limestone below the trash rack and the U-shaped Channel of the HLOW. At the auxiliary spillway, slaking of these units will eventually undermine the limestone, which will fall onto the slope downstream of the ogee crest on the left side of the spillway.

(2) Recommendations. The slopes containing weather sensitive materials should be monitored during periodic inspections. Although not anticipated, if slaking becomes a problem, a covering of shotcrete or other form of protection may be required at the left abutment of the RCC dam. At the auxiliary spillway, continued slaking of the shale will not interfere with the function of the structure. Any falling blocks of limestone will not damage the massive concrete of the sill and ogee. In addition, any interference of flows over the ogee crest would be relatively minor. If needed, however, debris could be easily removed.

d. Seismicity.

(1) As discussed in Paragraph 2.04a(3)(vi), a northeast-trending, range-bounding fault or zone of en echelon faults may extend through the auxiliary spillway. Range-bounding rift related faults are considered capable of continued faulting during seismic events. Any rupture along this fault zone during an earthquake would result in normal dip-slip movement. If offset was significant, it could result in cracking normal to the axis of the ogee crestof the auxiliary spillway.

(2) No chaotic features were observed in the syncline exposed in the spillway, but other exposures are brecciated, with some being well healed and mineralized. This indicates that intermittent movement has occurred along different segments of this fault over time. Based on the observed deformation of the Pliocene Palomas gravel, the most recent movement along this fault can be assumed to be 4± million years or less. It is, however, traceable as a lineament through possibly younger sediments to the northeast. It may not be possible to accurately date this fault zone.

(3) Recommendations. The monuments at the auxiliary spillway and RCC dam should be surveyed on a regular basis. After major regional seismic events, the monuments should be measured to determine if deformation along the fault is continuing. The chance of a seismic event during high pool is extremely remote, therefore, any damage to the auxiliary spillway as a result of a rupture along this fault would not likely interfere with the design function of this structure. Because design of the RCC dam was based on the criteria provided in the original Seismic Analysis Report, which did not identify this fault zone, the Corps of Engineers may need to re-evaluate the seismic risk for the project.

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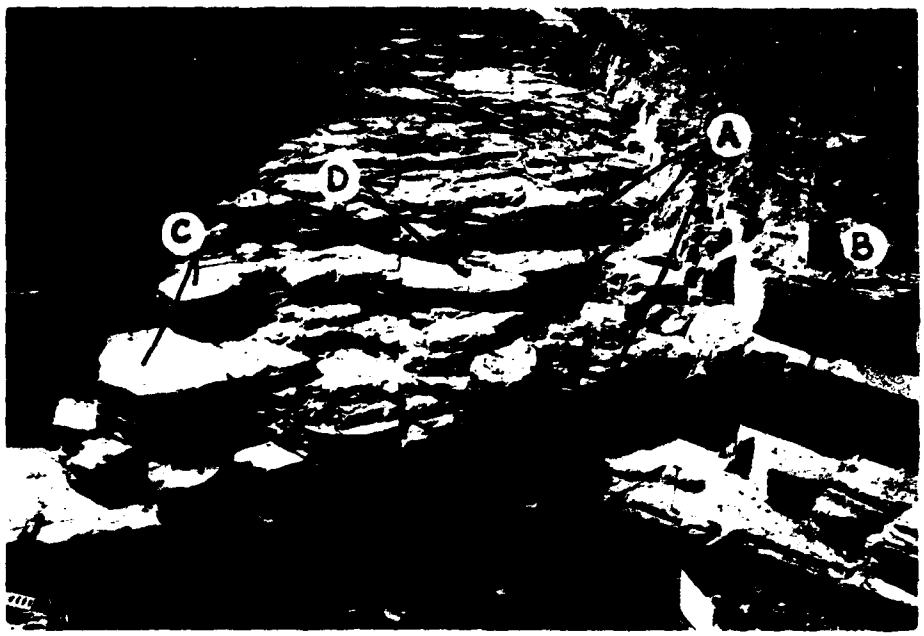


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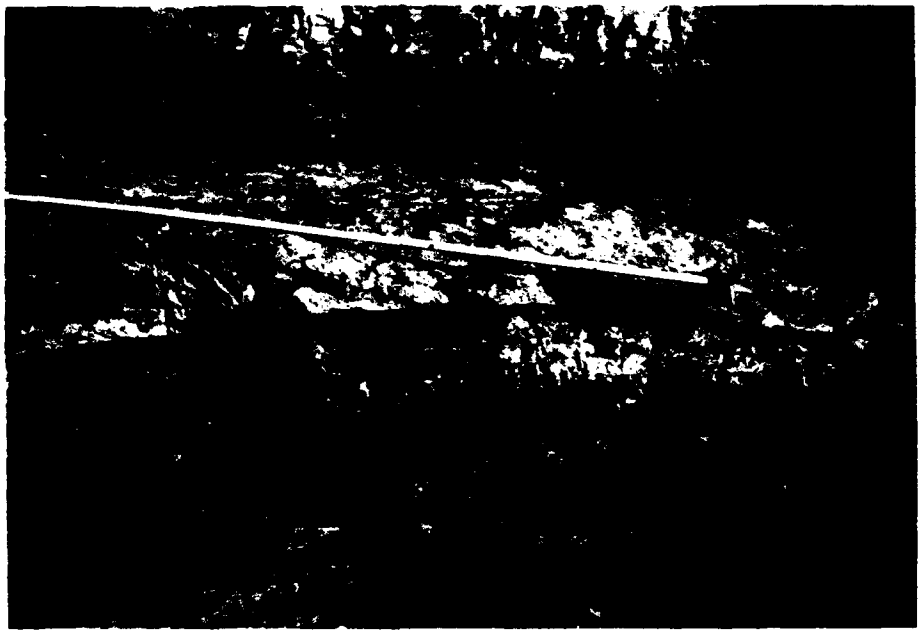


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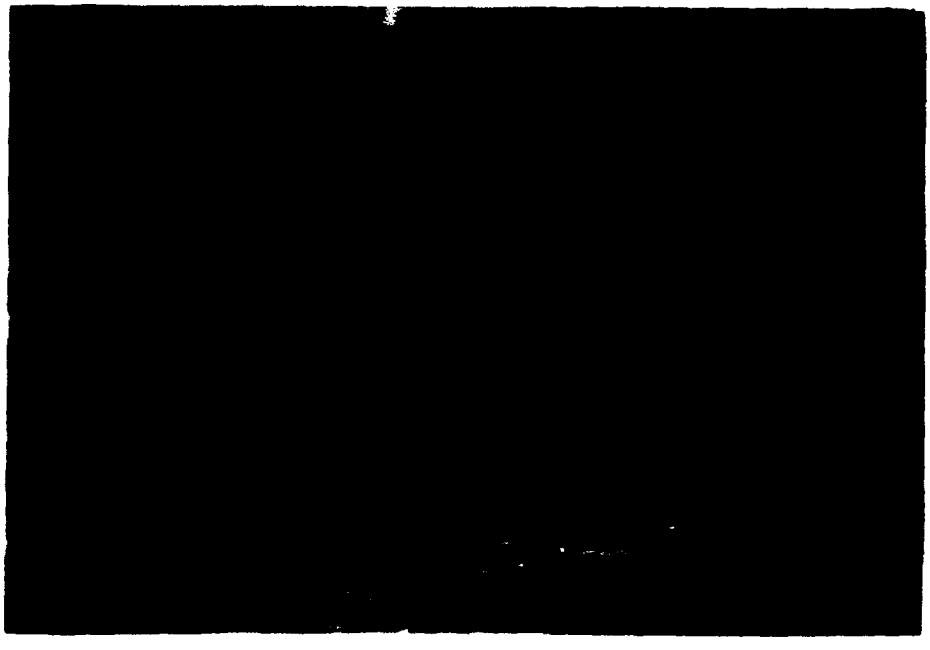


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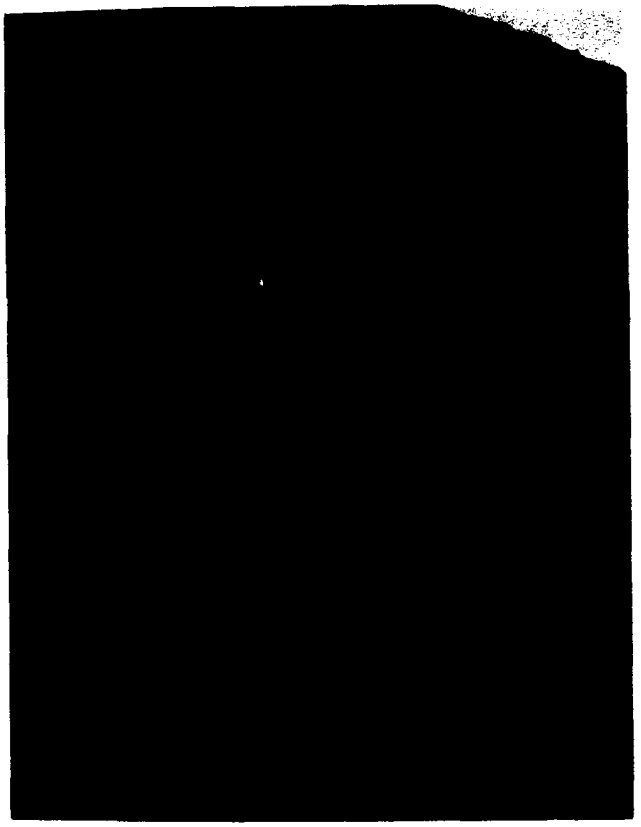


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Photo No. 13 - Looking northeast at the left side of the foundation for the HLOW at interbedded limestone, shale, and siltstone. 20 November 1991

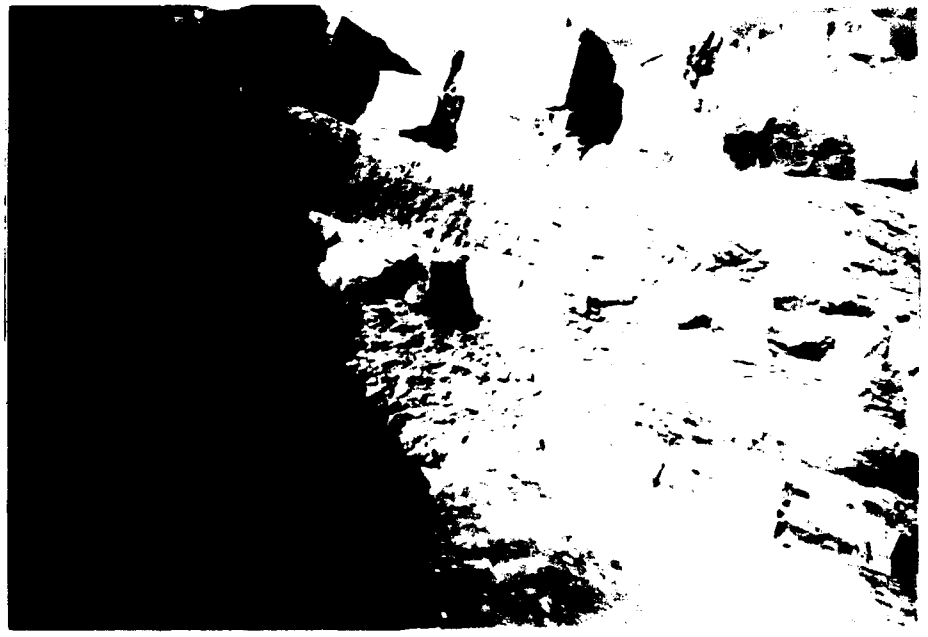


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Photo No. 15 - Looking north-northeast at the left abutment of the foundation trench illustrating freshly excavated shale. 6 March 1990



Photo No. 16 - Looking north-northeast at the left abutment of the foundation trench illustrating shale that has been exposed for several months. 24 August 1990



Photo No. 17 - Looking north at the left side of the auxiliary spillway illustrating an exposure of the Mud Mountain fanglomerate. 29 June 1990

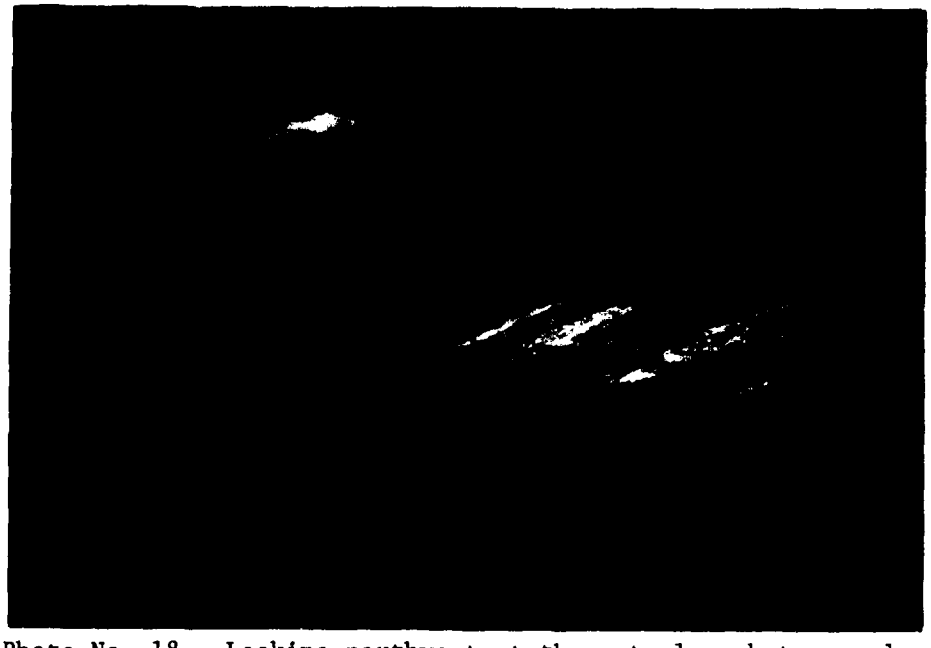


Photo No. 18 - Looking northwest at the cut slope between elev. 4,705 feet and 4,715 feet at the auxiliary spillway at exposed Palomas gravel units. 27 February 1990



Photo No. 19 - Looking north-northeast at the left abutment of the foundation trench illustrating severe alteration in and adjacent to joints and the shear/breccia zone. 6 March 1990



Photo No. 20 - Looking south-southeast at the left side of the auxiliary spillway illustrating severe alteration adjacent to joints. 12 November 1990



Photo No. 21 - Looking south-southeast along the left abutment of the foundation trench at solution cavities. 23 January 1991



Photo No. 22 - Looking southwest at the floor of the right abutment foundation trench illustrating alteration of the limestone adjacent to joints. 27 February 1990

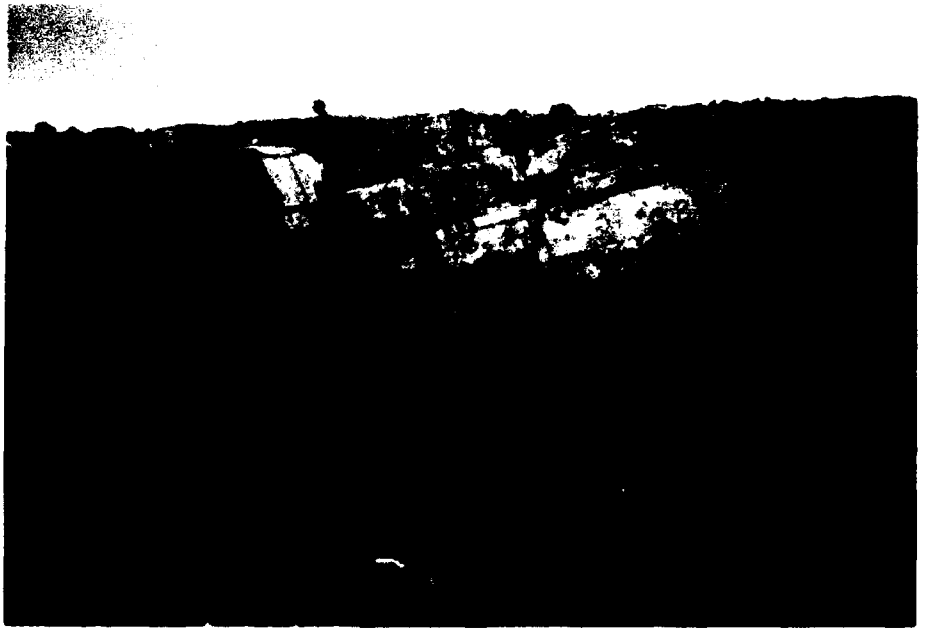


Photo No. 23 - Looking south at the downstream face of the right abutment foundation trench illustrating alteration of the limestone adjacent to joints. 27 February 1990



Photo No. 24 - Looking east at the right abutment foundation trench illustrating complex alteration pattern in the limestone. 15 August 1990



Photo No. 25 - Looking southeast along the LLOW foundation illustrating solutioning developed at intersection of two joints. 10 July 1990



Photo No. 26 - Looking northeast at the bottom and the left abutment foundation trench illustrating flooding. 24 July 1990



Photo No. 27 - Looking north-northeast at the bottom and left abutment foundation trench at alluvial material deposited by flood water. 4 September 1990



Photo No. 28 - Looking southeast at the channel bottom illustrating quicksand nature of saturated alluvium. Note buried excavator near center of photo. 12 October 1990



Photo No. 29 - Looking northeast at the left abutment foundation trench illustrating the 70-foot interval not encountered during pre-construction investigations. 5 April 1990

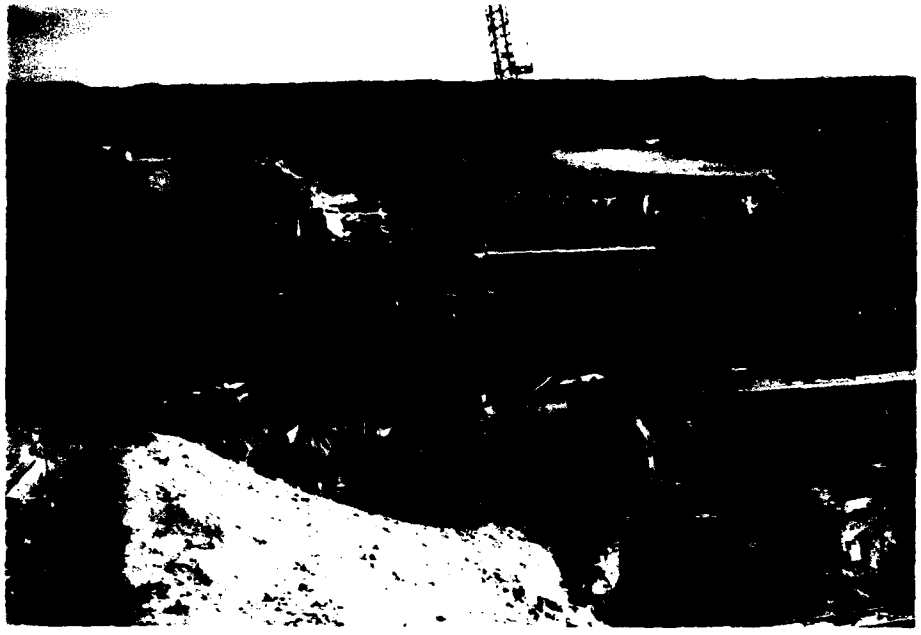


Photo No. 30 - Looking northeast at the left abutment foundation trench illustrating pumping dental concrete into solution cavity zone. 1 February 1991



Photo No. 31 - Looking east-southeast at HLOW foundation illustrating the volume of dental concrete required to form the foundation to design grade (top of wood formwork). 24 August 1990



Photo No. 32 - Looking north-northwest at the HLOW foundation illustrating poor quality rock on the right side (left in photo) of the discharge channel. 27 February 1990

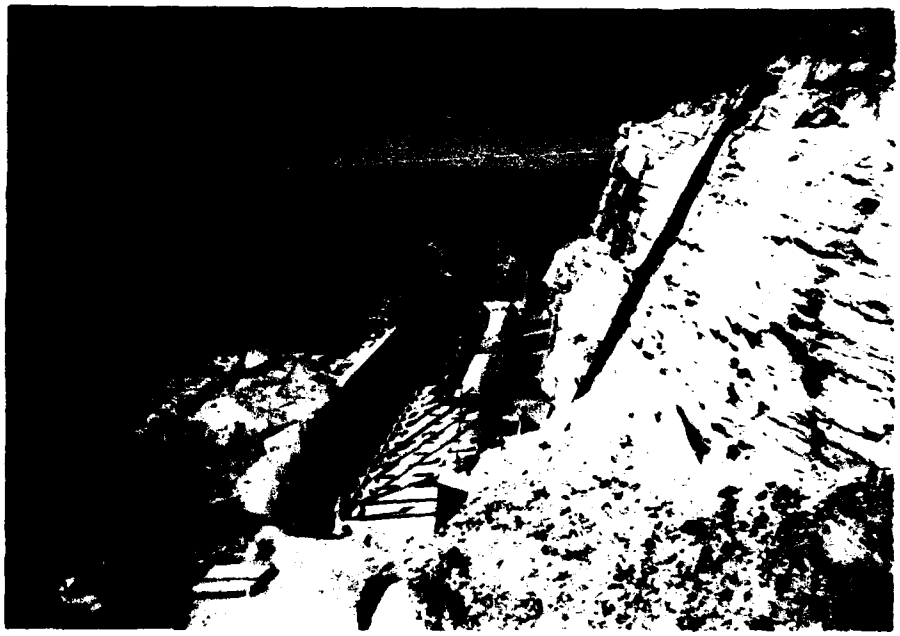


Photo No. 33 - Looking north-northwest at the HLOW illustrating construction of the "U"-shaped discharge channel after the wedge of rock was removed. 21 October 1990



Photo No. 34 - Looking northwest at the LLOW foundation illustrating foundation after initial cleaning. 29 June 1990

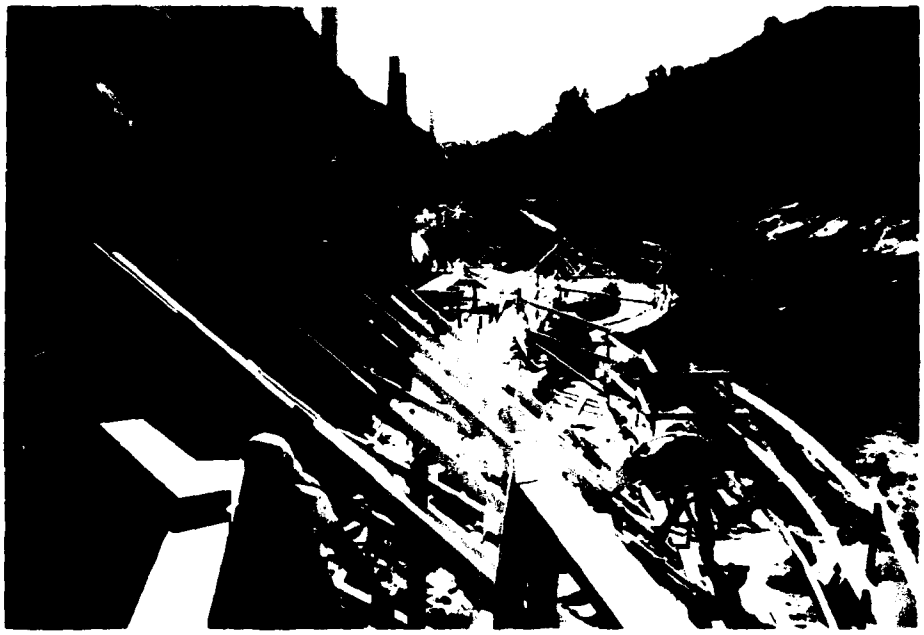


Photo No. 35 - Looking southeast along the LLOW illustrating formwork for the concrete encasement of the conduit. 30 October 1990



Photo No. 36 - Looking southwest at the right abutment foundation trench illustrating the removal of altered limestone and joint infilling material. 8 September 1990

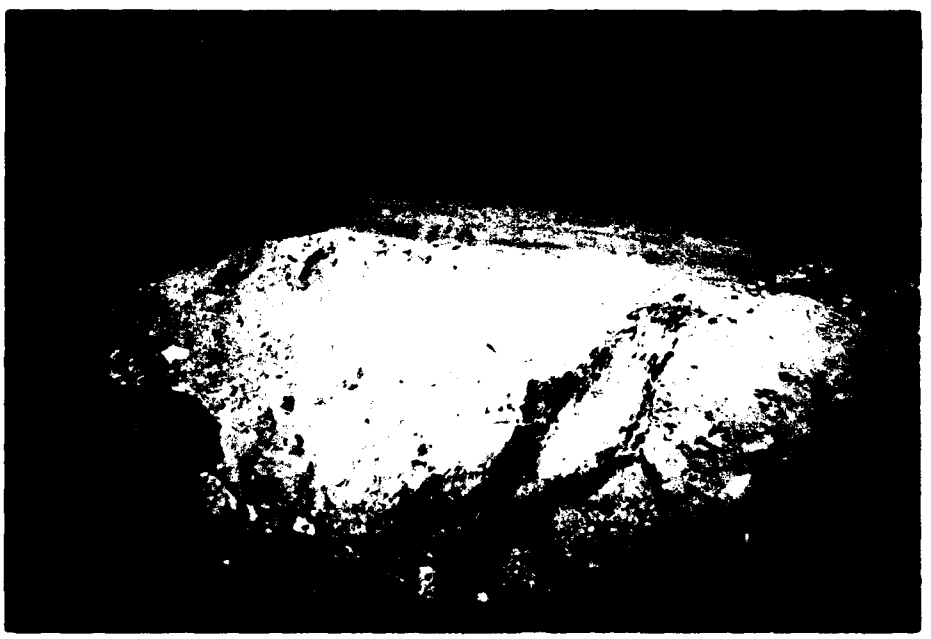


Photo No. 37 - Looking north at the auxiliary spillway illustrating a large boulder of consolidated conglomerate of the Palomas gravel. 26 January 1990



Photo No. 38 - Looking southeast at the auxiliary spillway illustrating the D-9's efforts to rip the consolidated conglomerate. 26 January 1990



Photo No. 39 - Looking south-southwest at the left side of the auxiliary spillway illustrating the solution cavities. 20 November 1990



Photo No. 40 - Looking northeast at the left side of the auxiliary spillway illustrating the placement of dental concrete into solution cavities. 4 December 1990



Photo No. 41 - Looking southeast at the left side of the auxiliary spillway illustrating the placement of dental concrete into solution cavities. 4 December 1990



Photo No. 42 - Looking southwest at the placement of RCC for the main dam illustrating the installation of control joints. 9 April 1991

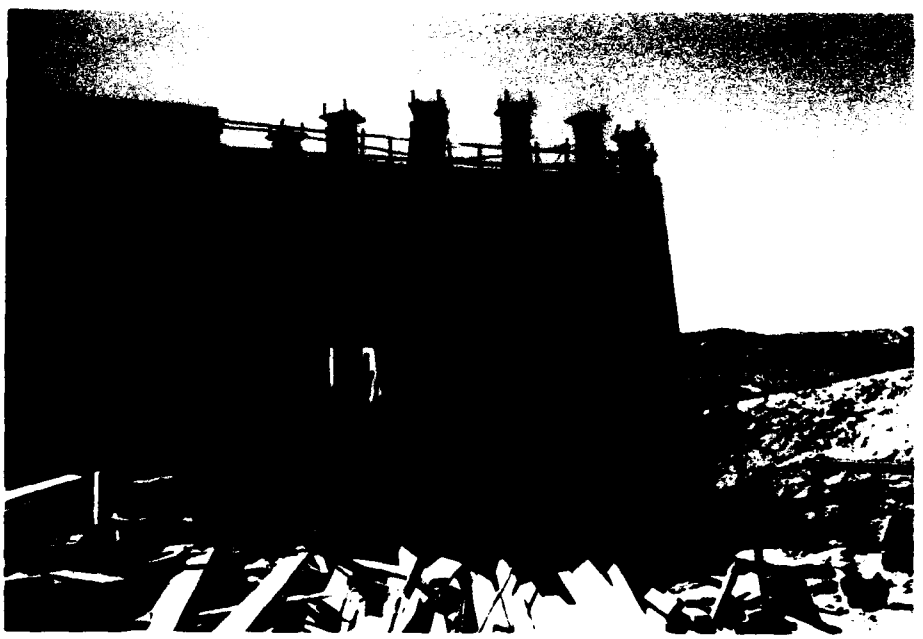


Photo No. 43 - Looking southeast at the HLOW intake illustrating relocated trashrack. Note the close proximity of the right side to the excavated slope. 29 March 1991

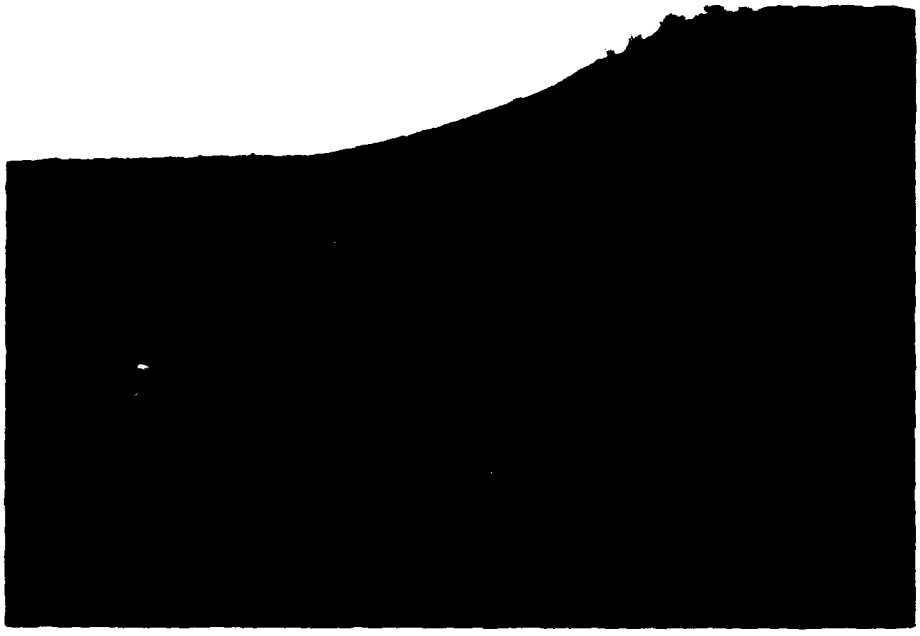


Photo No. 44 - Looking west at the right abutment of the auxiliary spillway illustrating the excavated cutoff trench. 24 July 1990



Photo No. 45 - Looking southeast along the earth embankment alignment illustrating the inspection trench. 9 February 1990

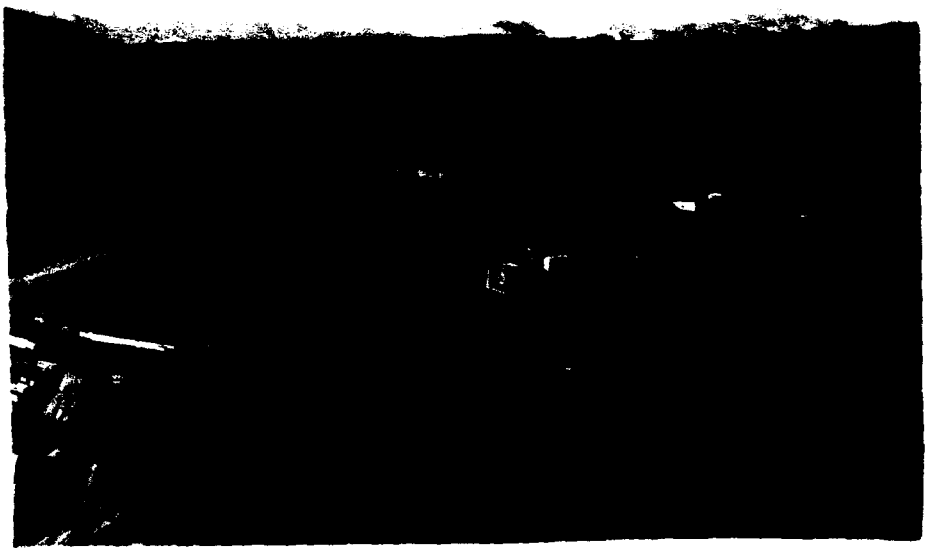


Photo No. 46 - Looking northwest at the office complex and borrow area illustrating the diversion channel and holding pond. 30 November 1991

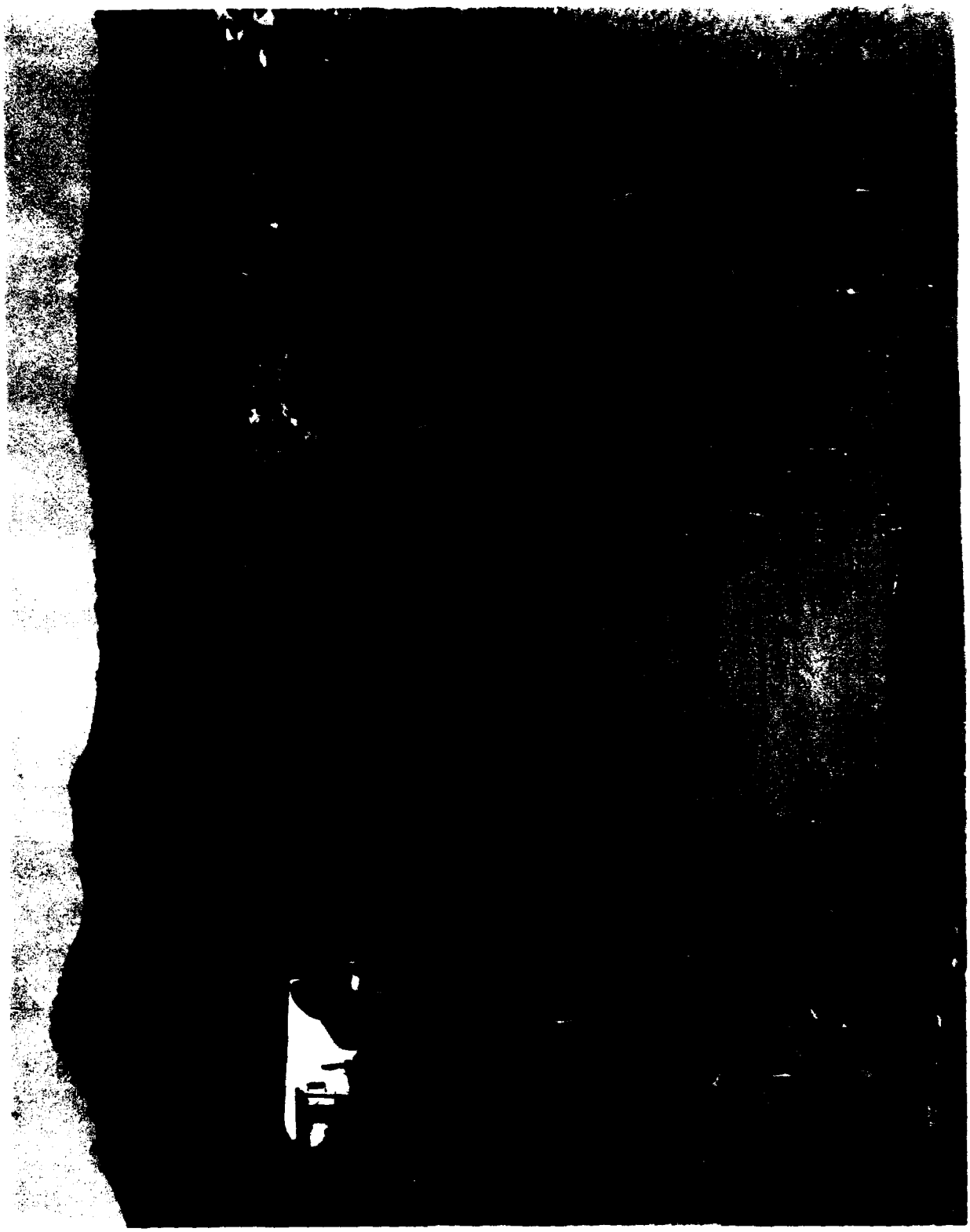


Photo No. 47 - Looking south at the borrow area illustrating flooding after the breach in the diversion channel. Note high water mark on stockpiles. 31 July 1990



Photo No. 48 - Looking west-southwest at the auxiliary spillway illustrating excavation progress. 24 January 1990



Photo No. 49 - Looking southwest along dam alignment at the right abutment foundation trench illustrating excavation progress. 26 January 1990



Photo No. 50 - Looking northwest at the bottom of the foundation trench illustrating excavation progress. 31 May 1990

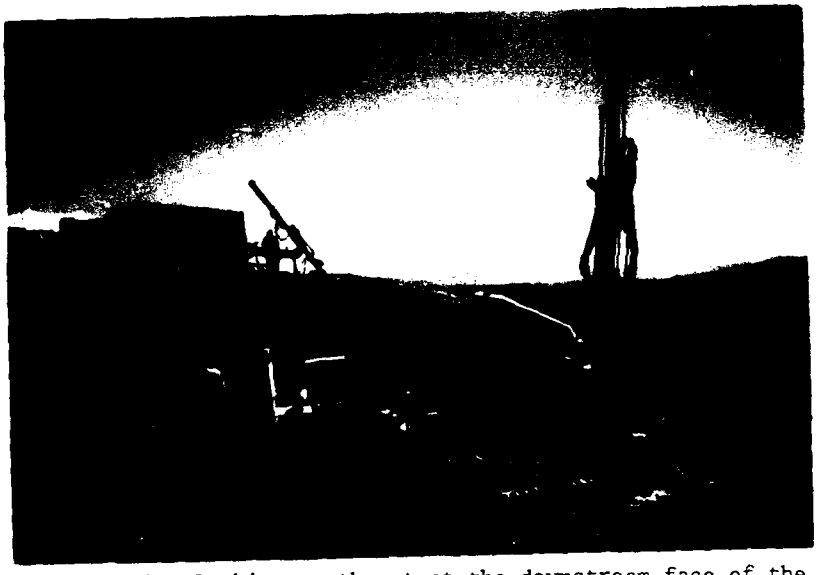


Photo No. 51 - Looking southeast at the downstream face of the foundation illustrating the Atlas Copco Hydraulic Track Drill. 9 February 1990



Photo No. 52 - Looking northeast at the auxiliary spillway illustrating a Gardner Denver Air Track Drill. 26 January 1990



Photo No. 53 - Looking up and northeast at the left abutment above the LLOW illustrating undercut and overhanging material. 21 October 1990



Photo No. 54 - Looking north-northwest at the left abutment of the foundation trench illustrating progress of excavation of the intermediate bench. Note drill rig performing Denison sampling. 15 December 1990



Photo No. 55 - Looking east at the left abutment foundation trench illustrating excavation of the intermediate bench nearing completion. 31 December 1990



Photo No. 56 - Looking southeast at the left abutment foundation trench illustrating drilling on intermediate bench.

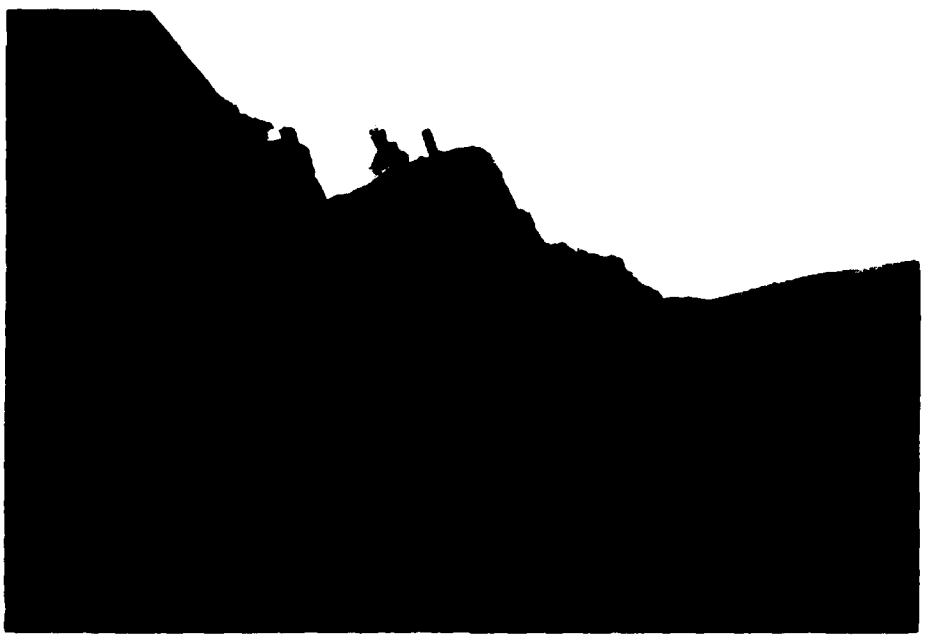


Photo No. 57 - Looking southeast at the left abutment foundation trench illustrating loading blast holes on the intermediate bench. 5 January 1991



Photo No. 58 - Looking down and southwest at the left abutment foundation trench illustrating the initial cleaning of cavities on the final bench. 15 January 1991



Photo No. 59 - Looking northwest at project site illustrating blast initiation. 4 December 1990



Photo No. 60 - Looking southwest at the left abutment foundation trench illustrating lowering the drill rig onto the HLOW for drilling the blast holes for the intermediate bench. 11 December 1990

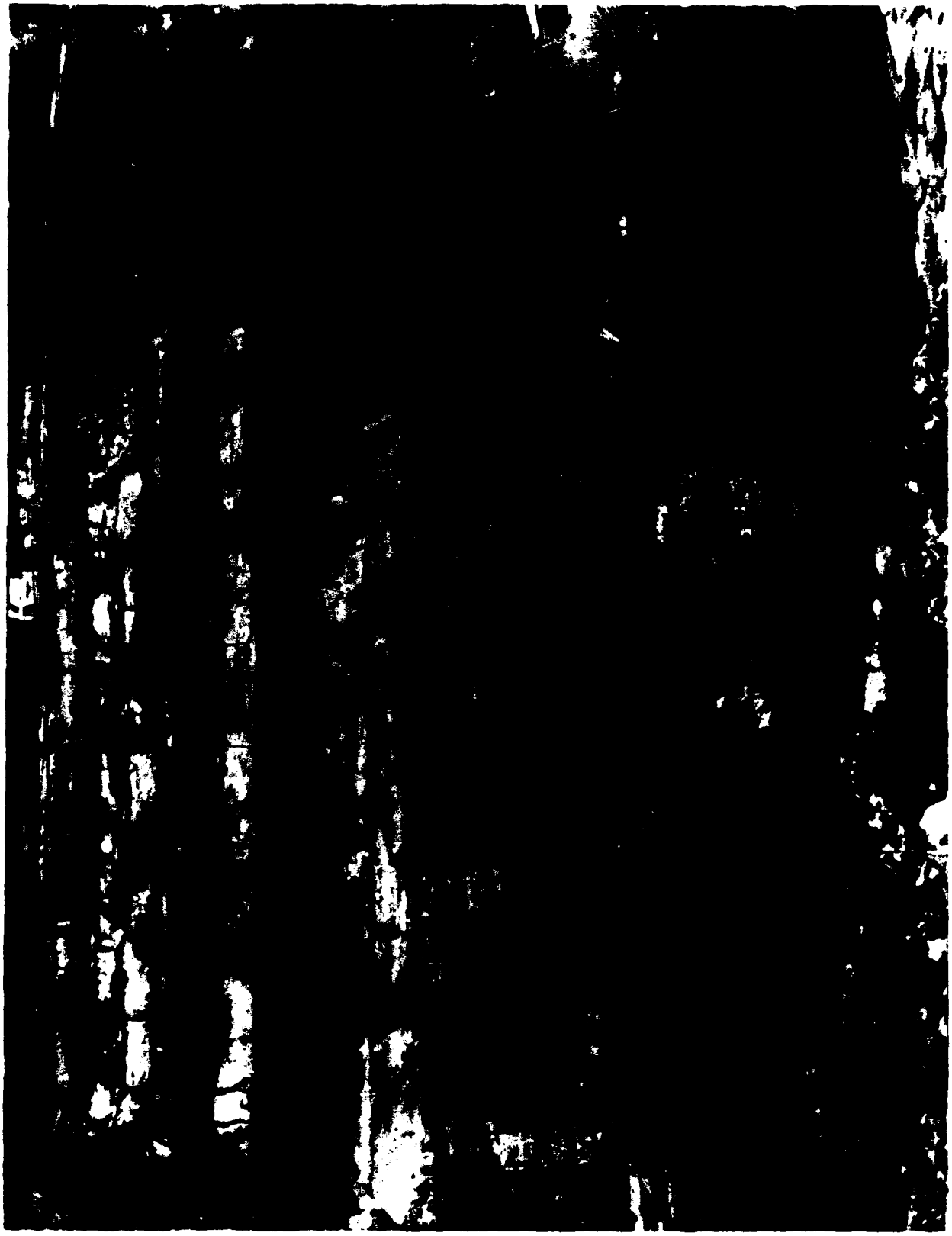


Photo No. 61 - Looking northeast at the left abutment foundation trench illustrating the sand protection for the LLOW conduit. 1 November 1990



Photo No. 62 - Looking northwest at the left abutment foundation trench illustrating the placement of blast mats. 31 December 1991



Photo No. 63 - Looking west at the auxiliary spillway illustrating vacuuming of the sill foundation. 4 July 1990



Photo No. 64 - Looking west-southwest at the chute toe foundation of the auxiliary spillway illustrating vacuuming of the foundation surface.



Photo No. 65 - Looking southeast at the left side of the auxiliary spillway illustrating the vacuum truck. 25 November 1990



Photo No. 66 - Looking southeast at the left side of the auxiliary spillway illustrating cleaning of breccia and gouge material. 6 July 1990

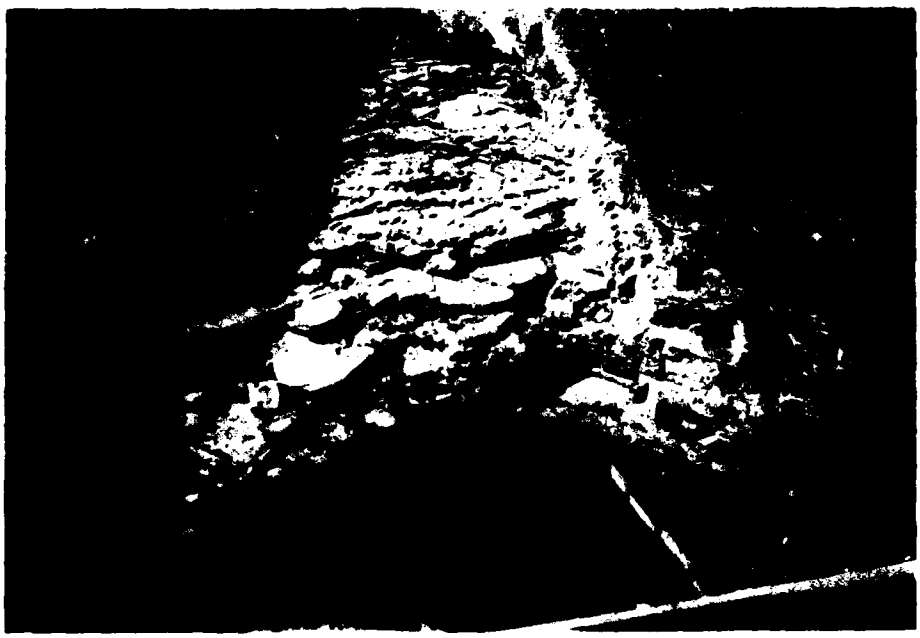


Photo No. 67 - Looking southwest along dam alignment to exposed right abutment foundation trench illustrating the "stairstep" surface. 12 March 1991



Photo No. 68 - Looking south at the left abutment foundation trench illustrating the "stairstep" surface. 8 June 1990



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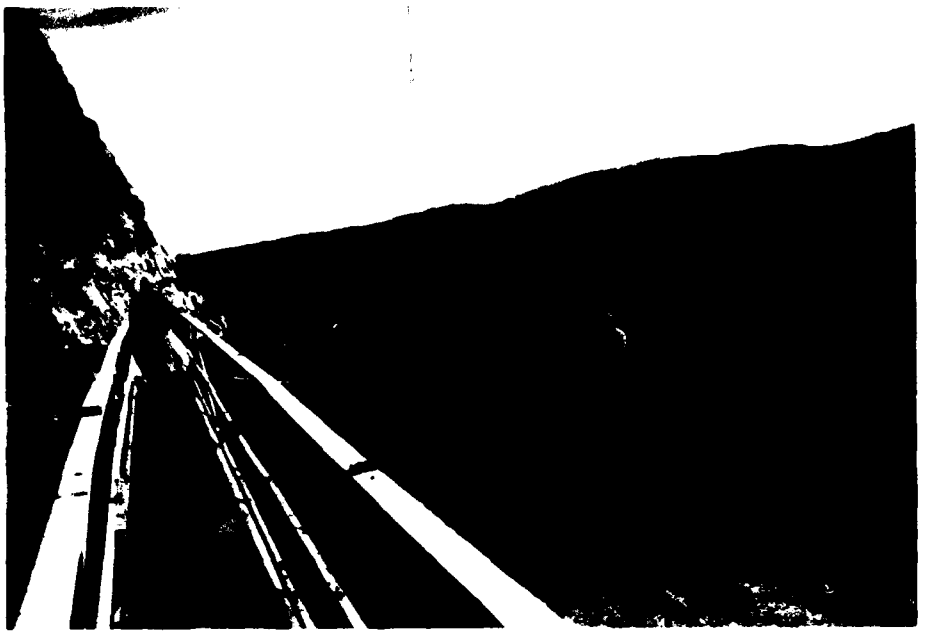


Photo No. 72 - Looking southeast at the left abutment foundation trench illustrating dental concrete placement into the solution cavities. 1 February 1991



Photo No. 73 - Looking northeast at the left abutment foundation trench illustrating cleaning of joints and cavities during RCC placement. 26 March 1991



Photo No. 74 - Looking up and northeast at the left abutment foundation trench illustrating the irregular surface of the slope below the LLOW. 8 February 1991

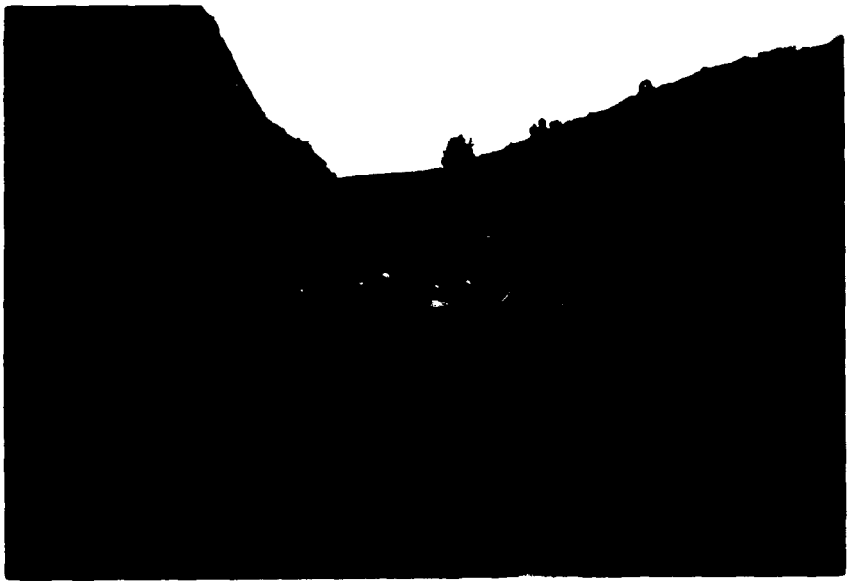


Photo No. 75 - Looking southeast along the alignment of the LLOW illustrating final cleanup before dental concrete placement. 4 September 1990

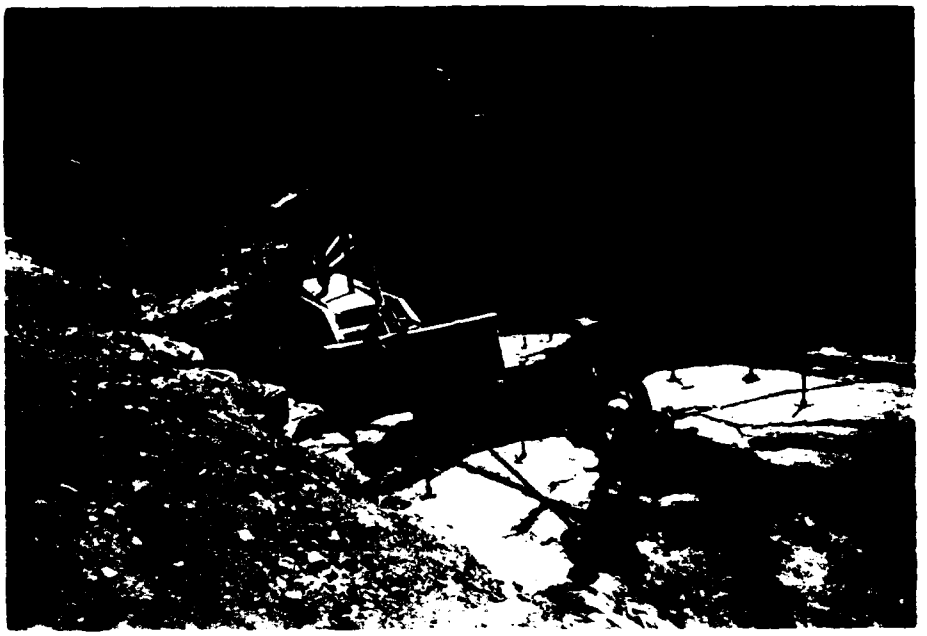


Photo No. 76 - Looking east-northeast at the right abutment foundation trench illustrating mechanical cleanup of the foundation. 8 September 1990



Photo No. 77 Looking northwest at the right abutment foundation trench illustrating hand cleaning of the foundation. 15 August 1990



Photo No. 78 - Looking northwest along the alignment of the HLOW illustrating the wire mesh. 5 October 1990



Photo No. 79 - Looking northwest along the alignment of the LLOW illustrating the wire mesh. 21 October 1990



Photo No. 80 - Looking east-northeast at the right abutment foundation trench illustrating installed rock bolts. 31 May 1990



Photo No. 81 - Looking northwest at the right abutment downstream of the dam illustrating the installed rock bolts. 4 February 1991

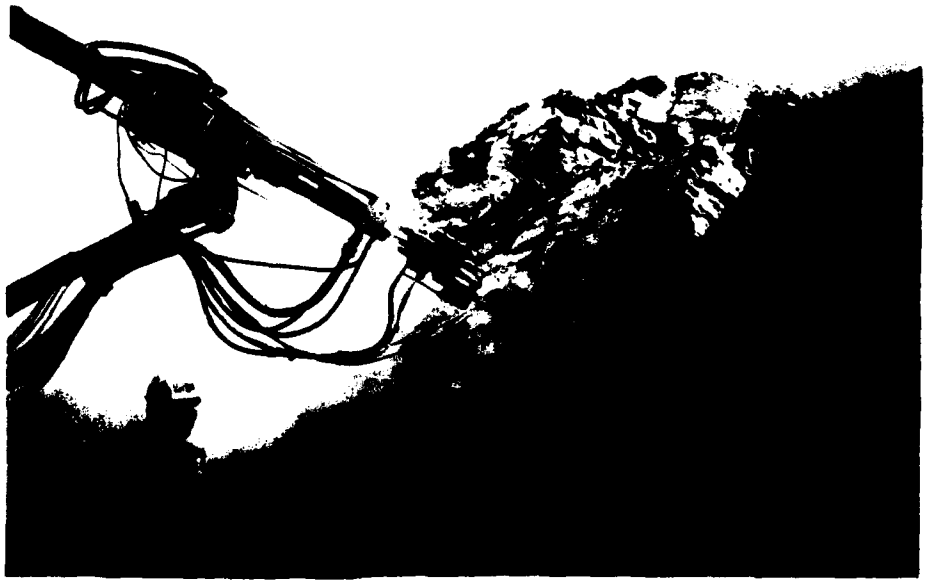


Photo No. 82 - Looking northwest at the left side of the HLOW illustrating drilling for rock bolts. 18 May 1990



Photo No. 83 - northwest at the left side of the HLOW illustrating the installation of a rock bolt. 18 May 1990



Photo No. 84 - Looking east at the left side of the HLOW illustrating turning a rock bolt using a pneumatic drill. 18 May 1990



Photo No. 85 - Looking northwest at the left side of the HLOW illustrating pull out testing of a rock bolt. 18 May 1990



Photo No. 86 - Looking west-southwest at the auxiliary spillway illustrating the final excavated surface. 21 April 1990



Photo No. 88 - Looking southeast at the bottom of the foundation trench illustrating the pumper truck placing dental concrete. 12 February 1991



Photo No. 87 - Looking northwest at the HLOW foundation illustrating the placement of dental concrete using the crane and bucket. 25 August 1990



Photo No. 89 - Looking northeast along the dam alignment illustrating the curing protection for the dental concrete. 29 August 1990



Photo No. 90 - Looking west-southwest at placement of RCC illustrating wetting of RCC and the foundation. Note the two laborers with garden hoses. 29 March 1991

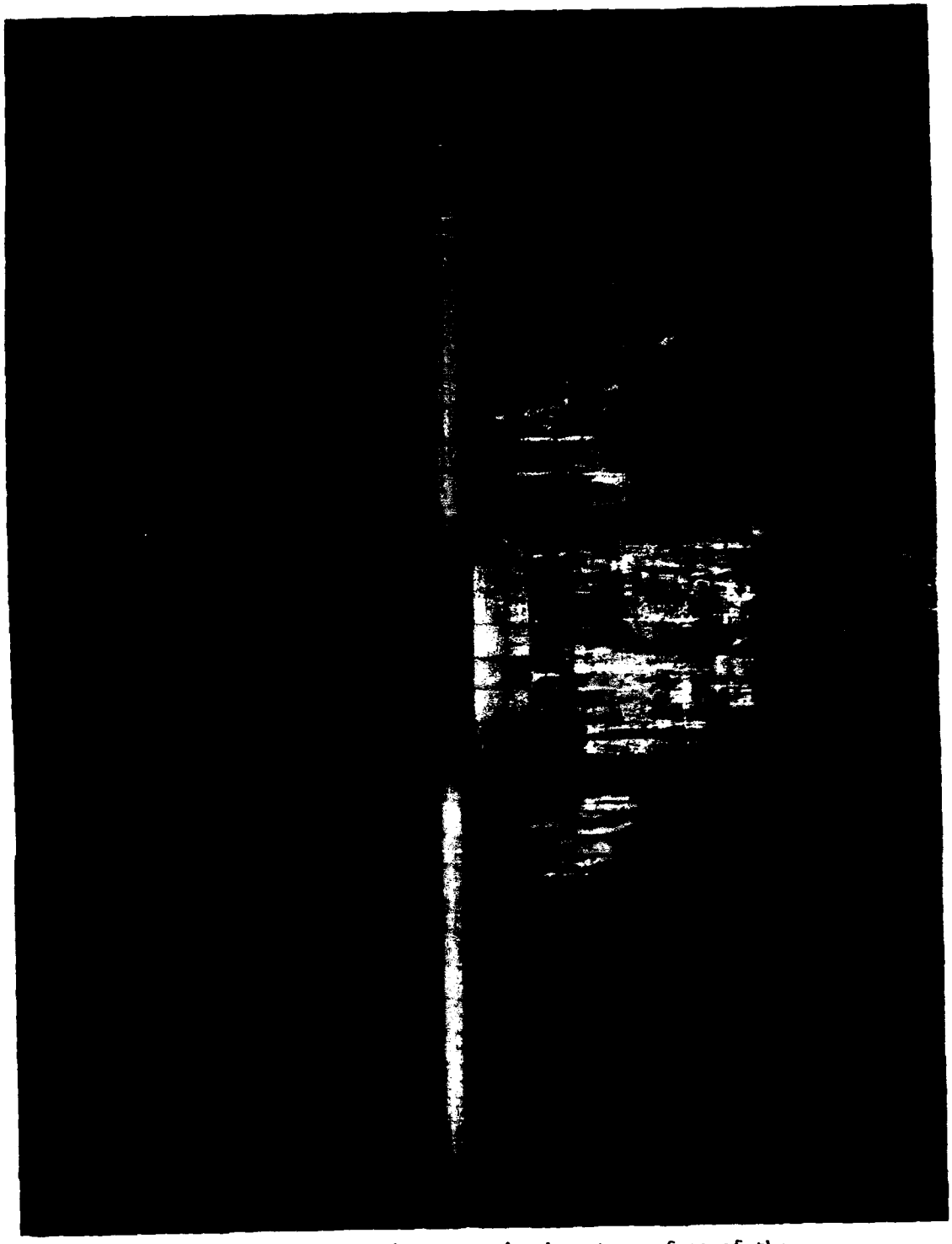


Photo No. 91 - Looking northwest at the downstream face of the completed RCC dam. 11 July 1991

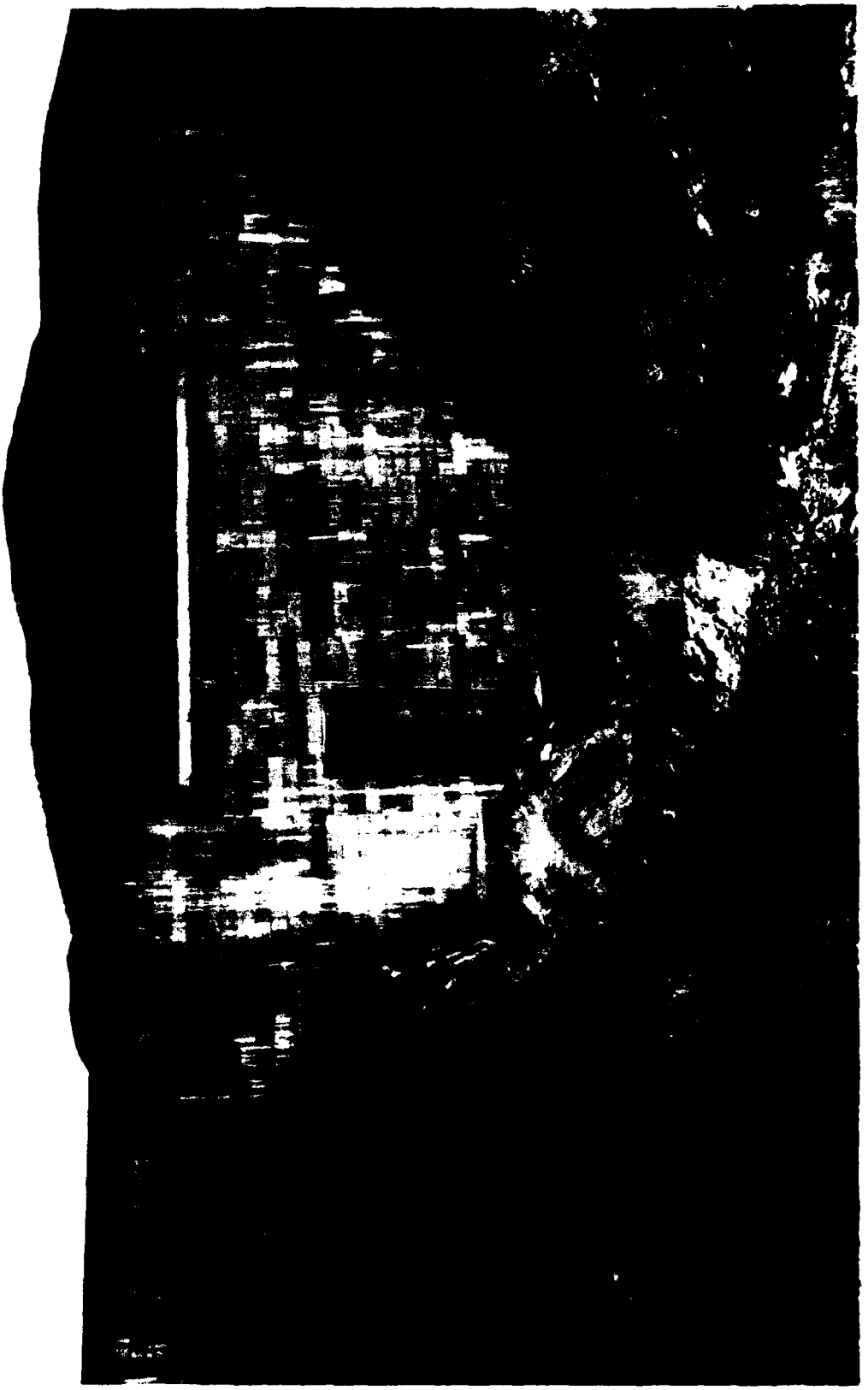


Photo No. 92 - Looking southeast at the upstream face of the completed RCC dam. 11 July 1991

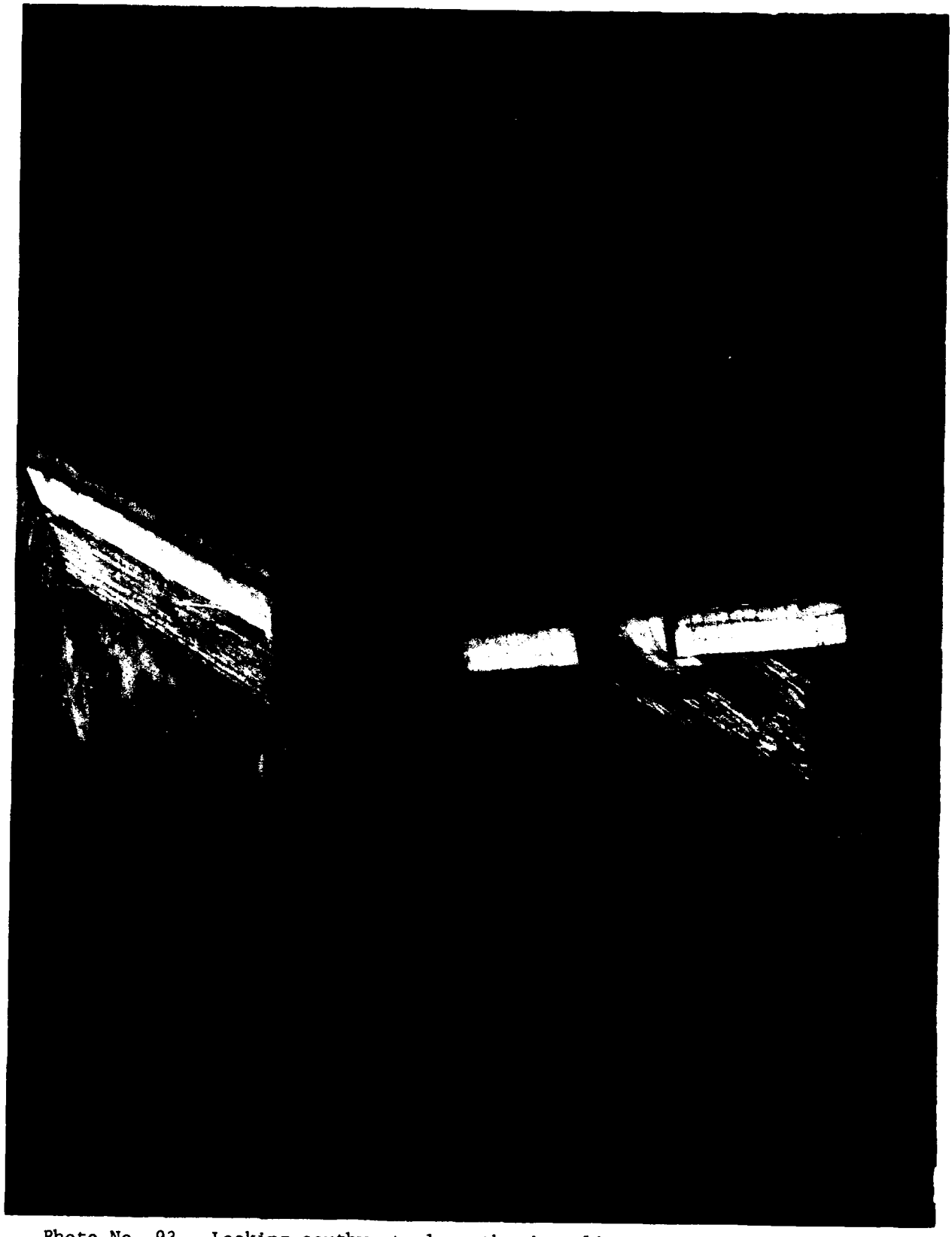
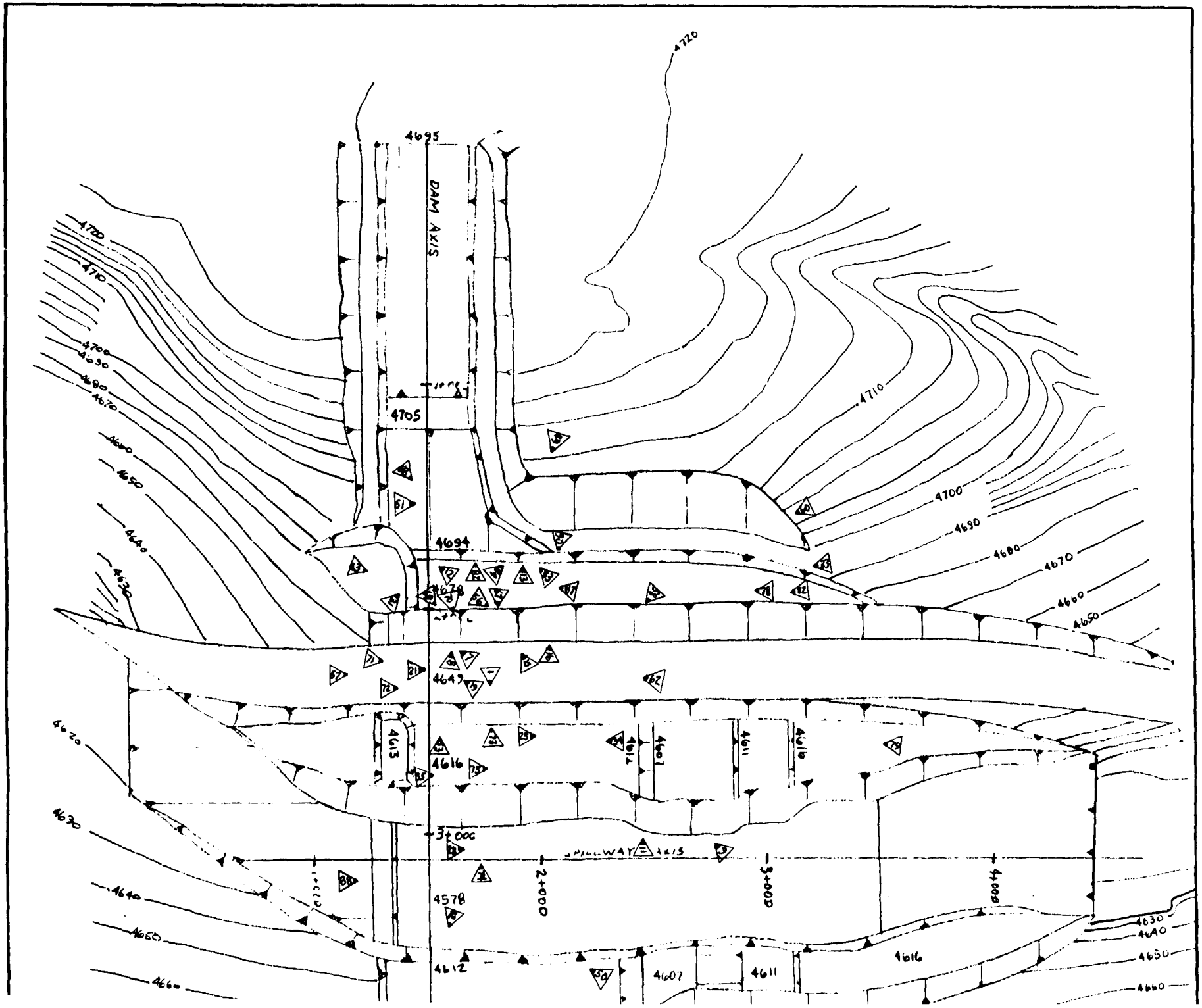
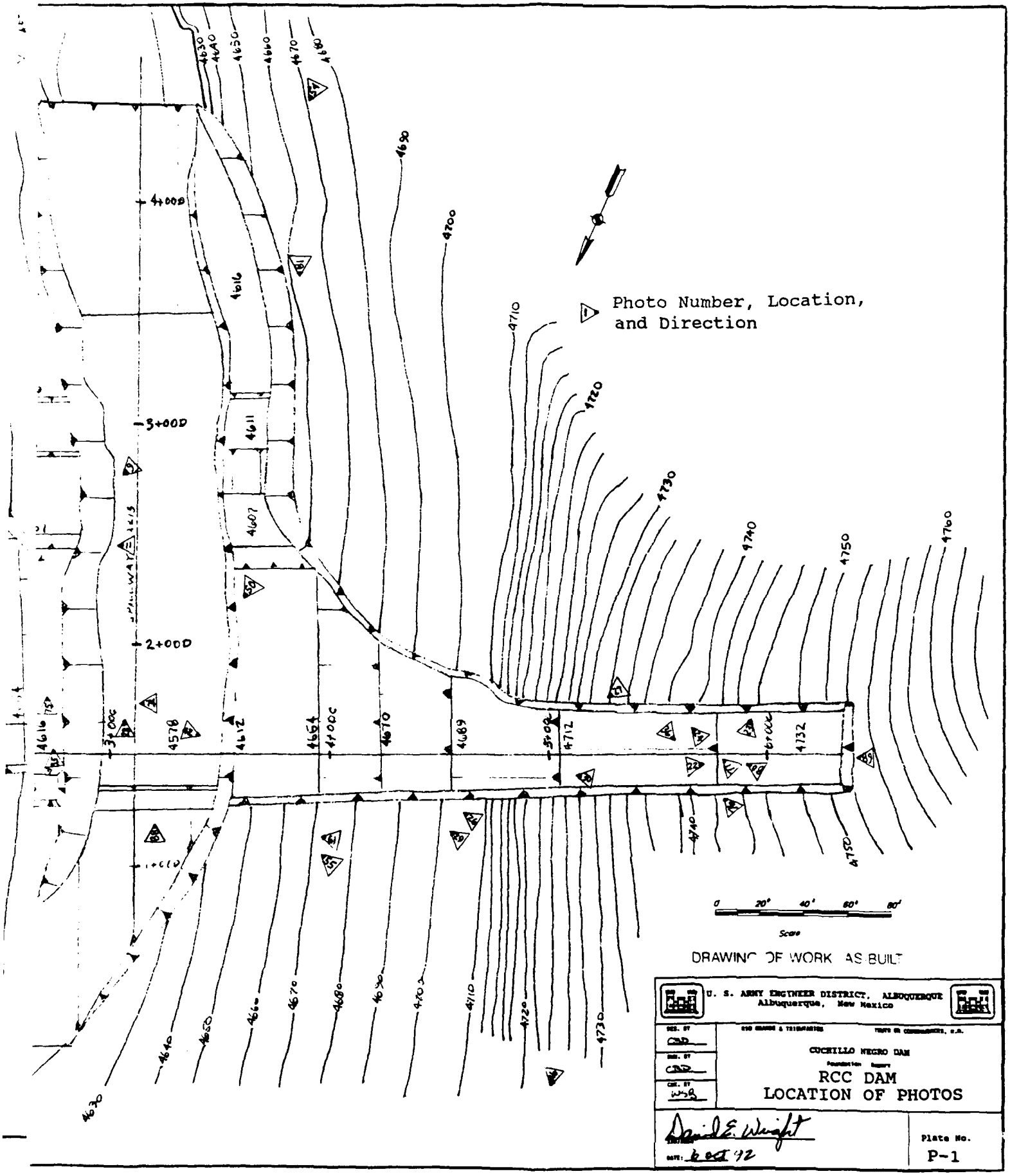


Photo No. 93 - Looking southwest along the dam alignment at the completed project. 11 July 1991


DAM AXIS
4695
4705
4694
4678
4649
4616
4613
4614
4607
4611
4610
4578
4612
SPILLWAY
1+000
2+000
3+000
4+000
4720
4710
4700
4690
4680
4670
4660
4650
4640
4630
4620



DRAWING OF WORK AS BUILT

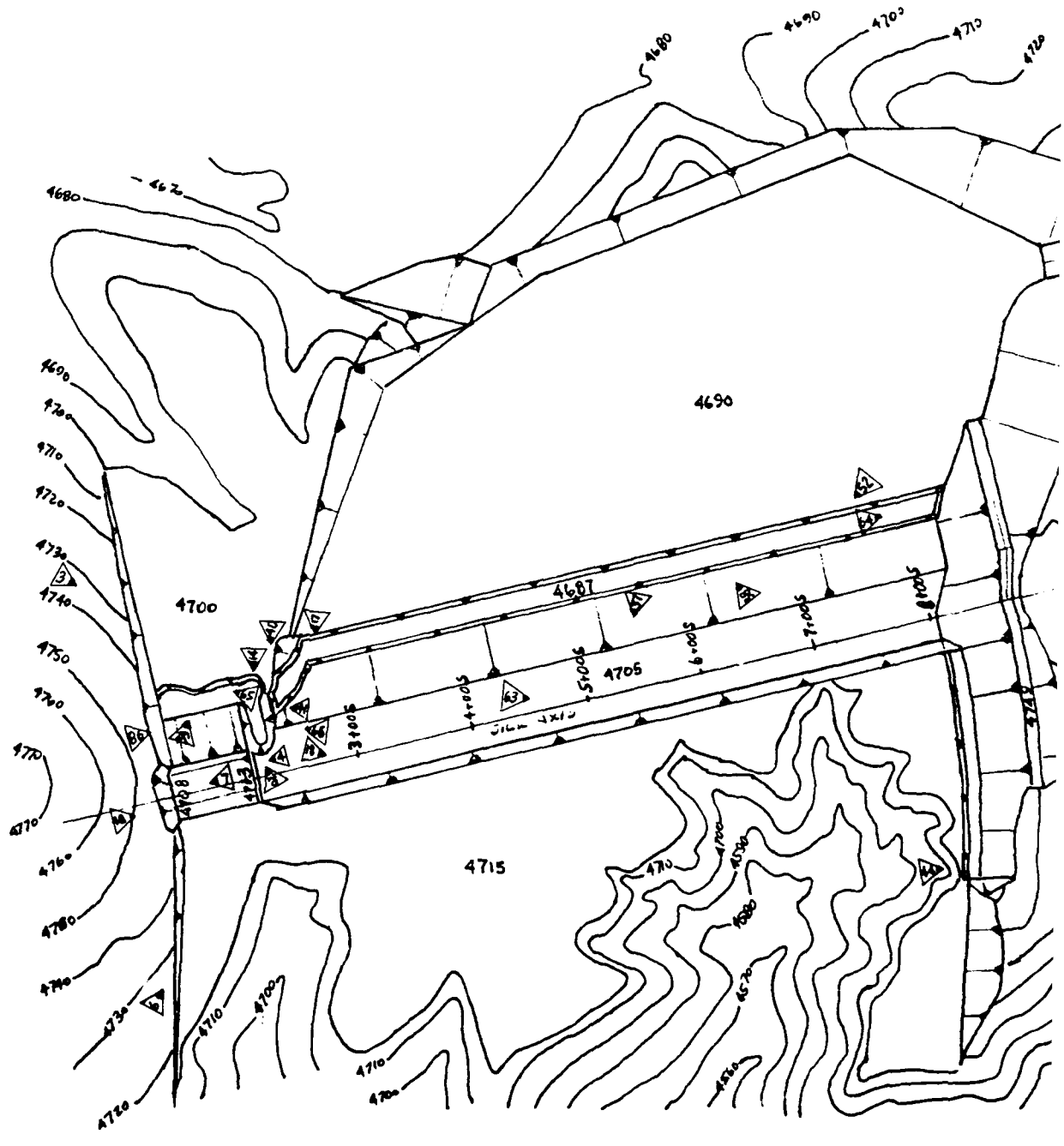
U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
Albuquerque, New Mexico

DES. BY
DRN. BY
CHK. BY

CUCHILLO NEGRO DAM
Foundation Report
RCC DAM
LOCATION OF PHOTOS

David E. Wright
DATE: Oct 92

Plate No.
P-1


4680
4690
4700
4710
4720
4730
4740
4750
4760
4770
4780
4790
4700
4705
4715
4687
3+00S
4+00S
5+00S
6+00S
7+00S
8+00S

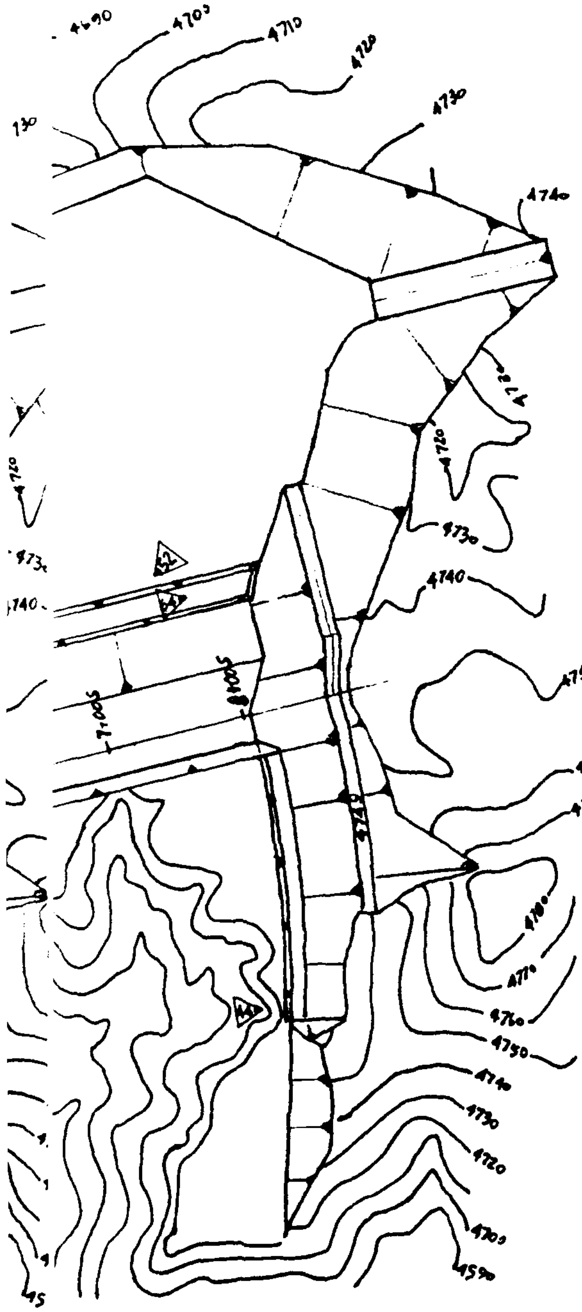
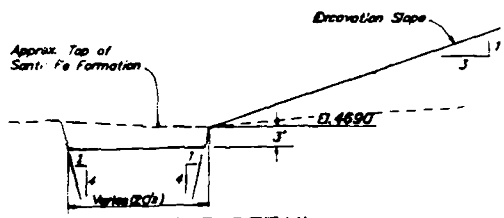


Photo Number, Location, and Direction



CHUTE TOE DETAIL

Scale: 1" = 25'

Scale: 0 50' 100' 150' 200'

DRAWING OF WORK AS BUILT

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
Albuquerque, New Mexico

COCHITI NEGRO DAM

AUXILIARY SPILLWAY
LOCATION OF PHOTOS

David E. Wright
DATE: 6 Oct 92

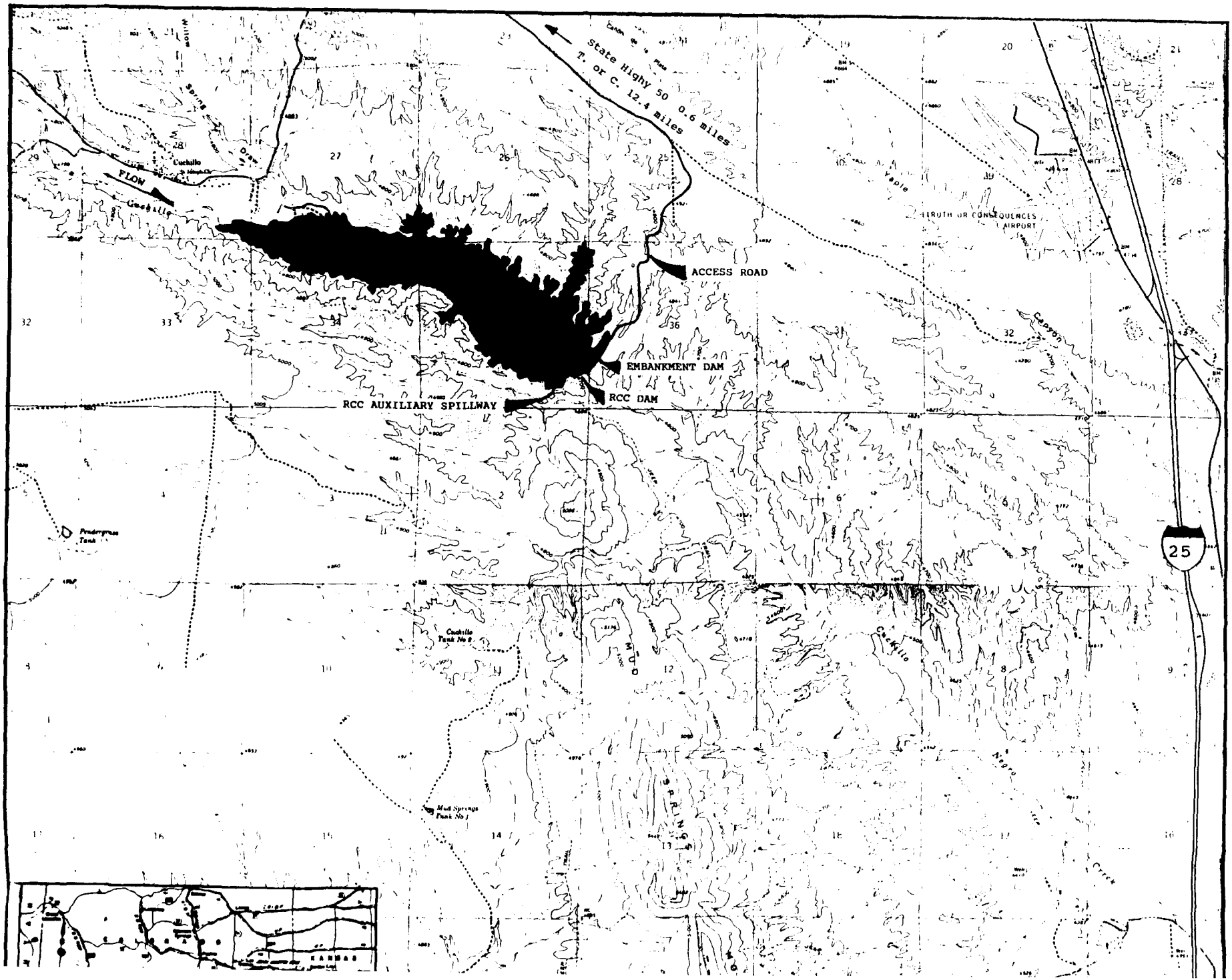
Plate No.
P-2

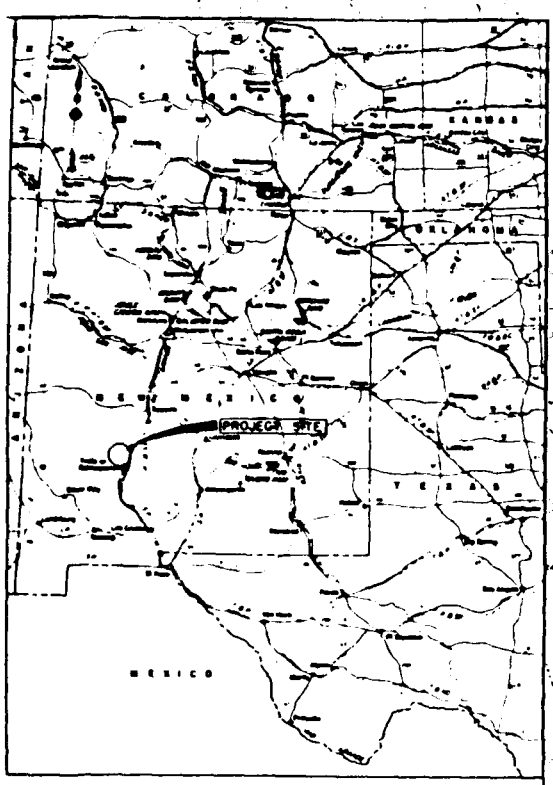
APPENDIX A

APPENDIX A

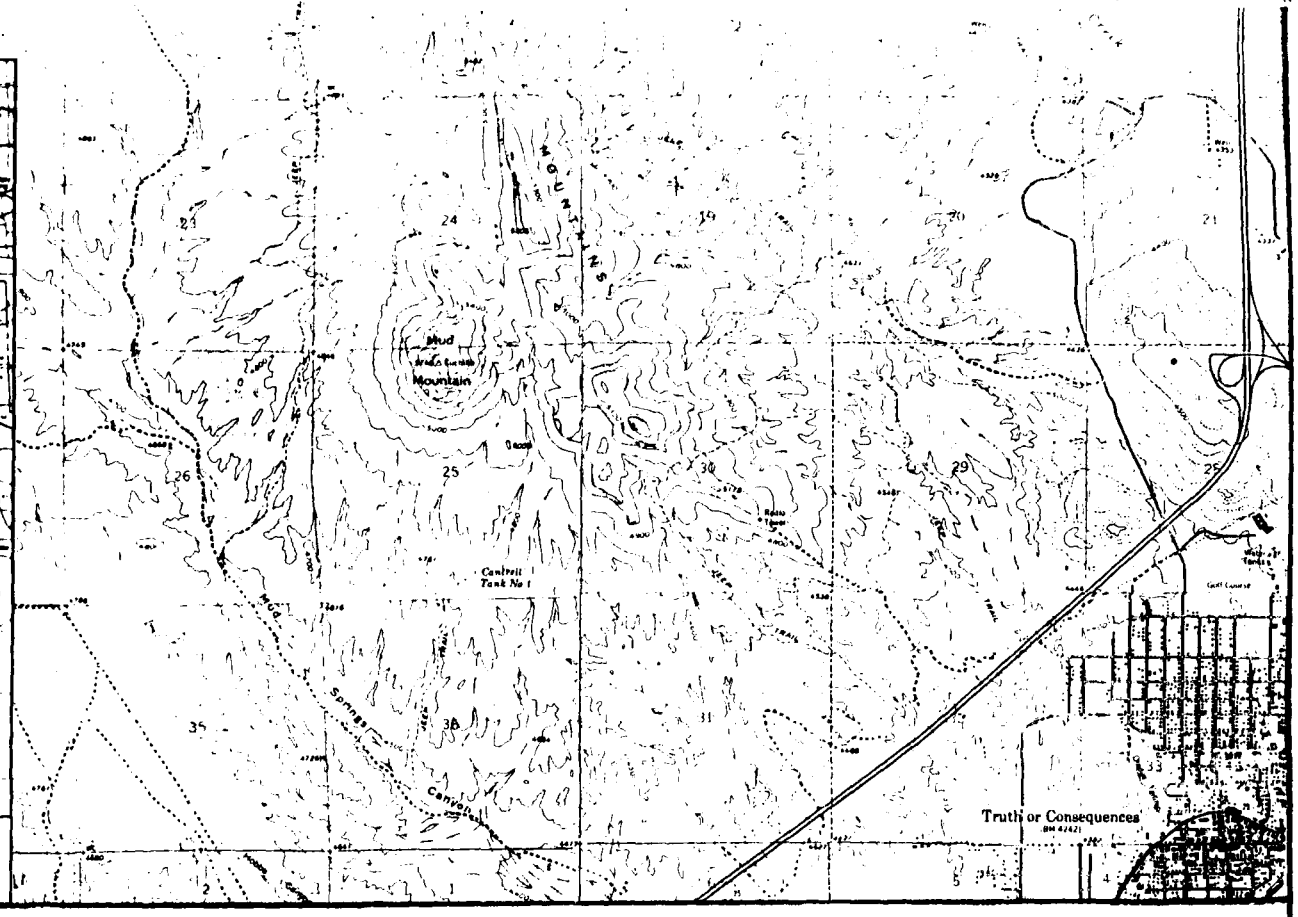
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State Highway 50
T. or C. 12.4 miles
0.6 miles
ACCESS ROAD
EMBANKMENT DAM
RCC DAM
RCC AUXILIARY SPILLWAY
FLOW
Cuchillo
Spring Creek
Vado
TRUTH OR CONSEQUENCES AIRPORT
Cuchillo Negro Creek
Cuchillo Tank No 2
Mud Springs Tank No 1
Pridgeon Tank
Negro Creek
25



VICINITY MAP



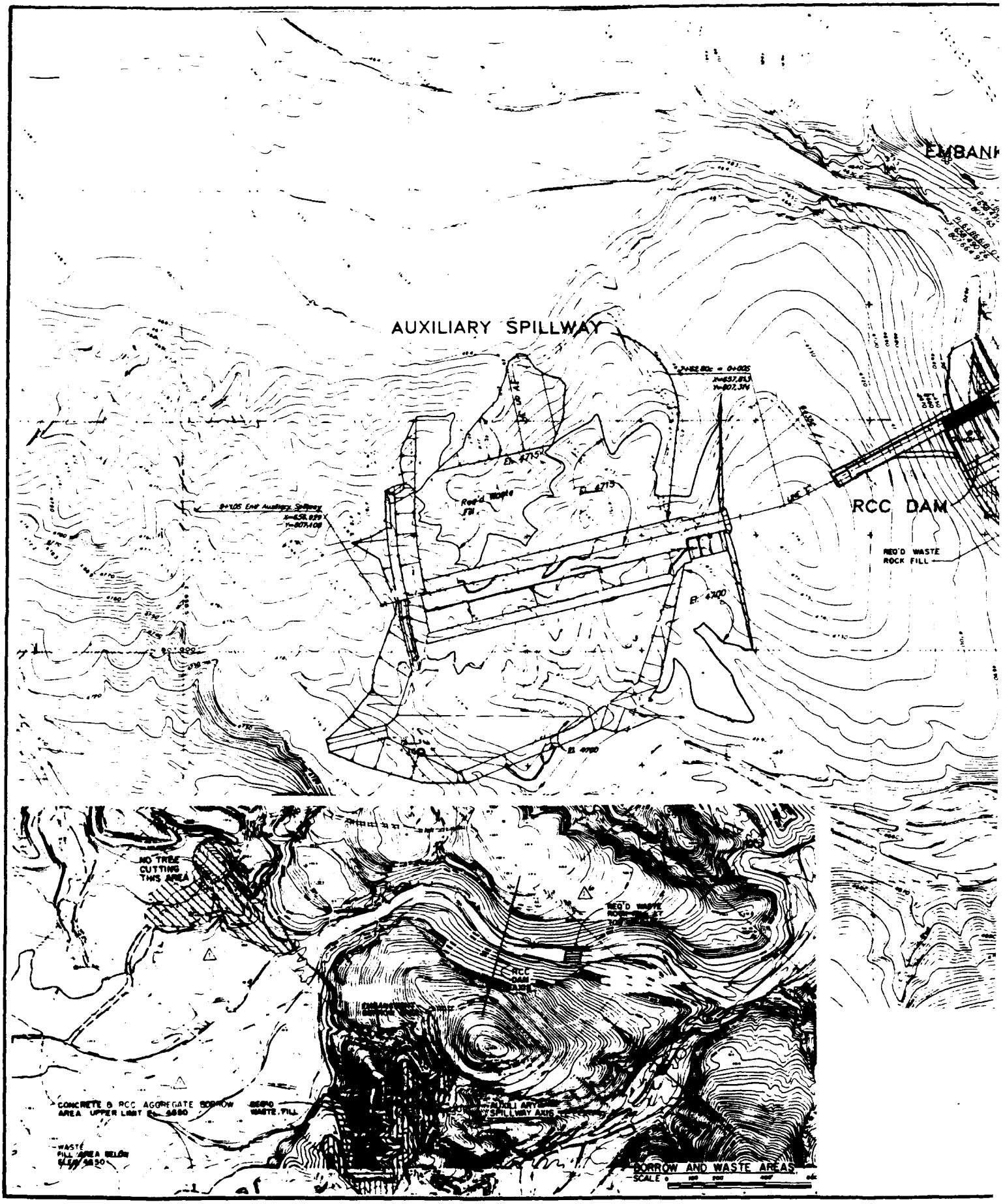
SCALE

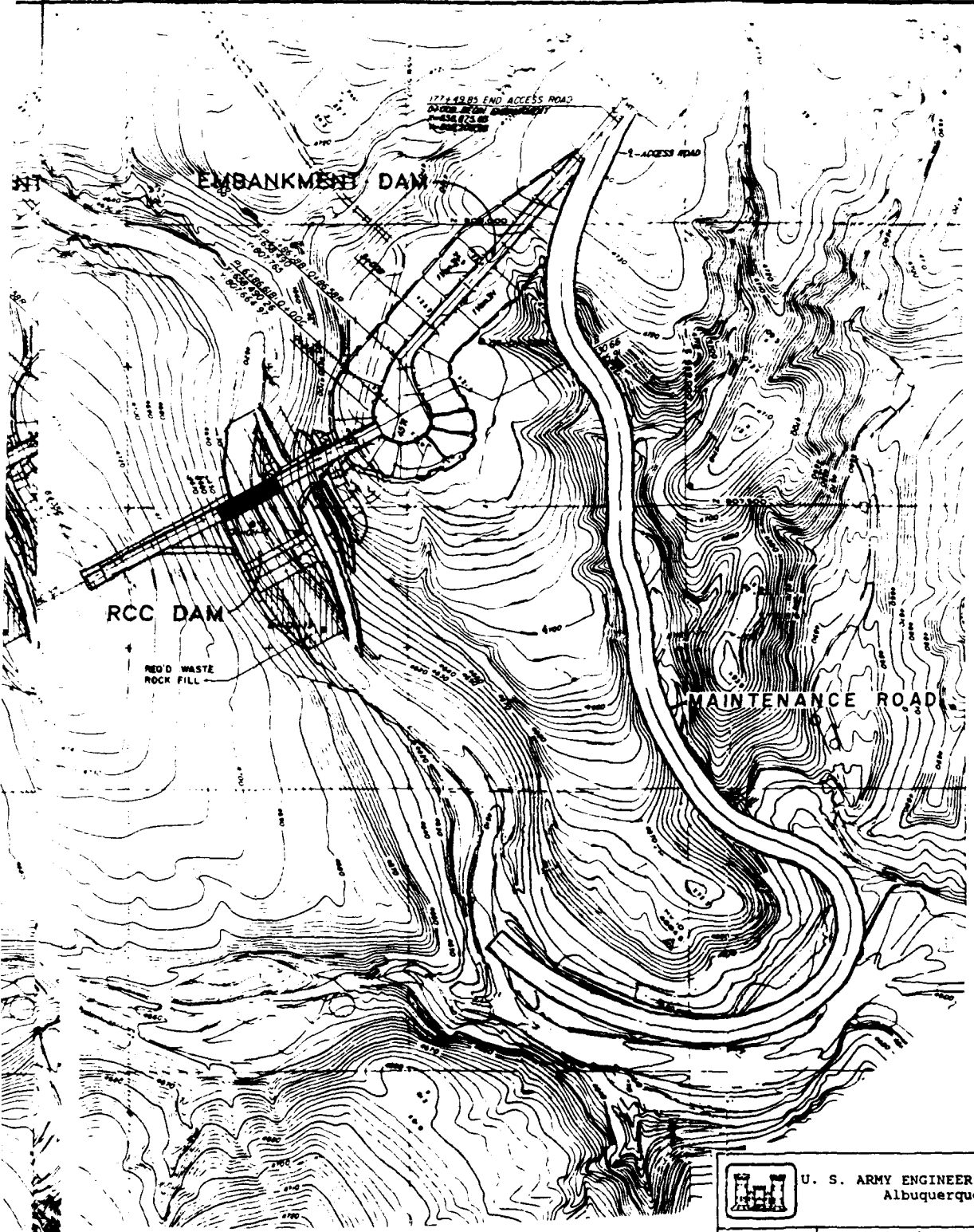
CONTOUR INTERVAL 20 FEET
NATIONAL GEODETIC VERTICAL DATUM OF 1929

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
Albuquerque, New Mexico

DES. BY CBD	RIO GRANDE & TRIBUTARIES — TRUTH OR CONSEQUENCES, N.M.
DWN. BY CBD	CUCHILLO NEGRO DAM
CHK. BY DEL	Foundation Report
	SITE & VICINITY MAP
David E. Wright ENGINEER DATE: 9-4-92	Plate No. A-1

A-2


AUXILIARY SPILLWAY
RCC DAM
REQ'D WASTE ROCK FILL
NO TREE CUTTING THIS AREA
RCC DAM AXIS
CONCRETE & RCC AGGREGATE BORROW AREA UPPER LIMIT EL. 5890
WASTE FILL AREA BELOW EL. 5830
BORROW AND WASTE AREAS



U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
Albuquerque, New Mexico

DES. BY CBD	RIO GRANDE & TRIBUTARIES	TRUTH OR CONSEQUENCES, N.M.
DWN. BY CBD	CUCHILLO NEGRO DAM Foundation Report	
CHK. BY DEL	PROJECT LAYOUT PLAN AND BORROW AND WASTE AREAS	
ENGINEER David E. Wright DATE: 9-4-92		Plate No. A-2

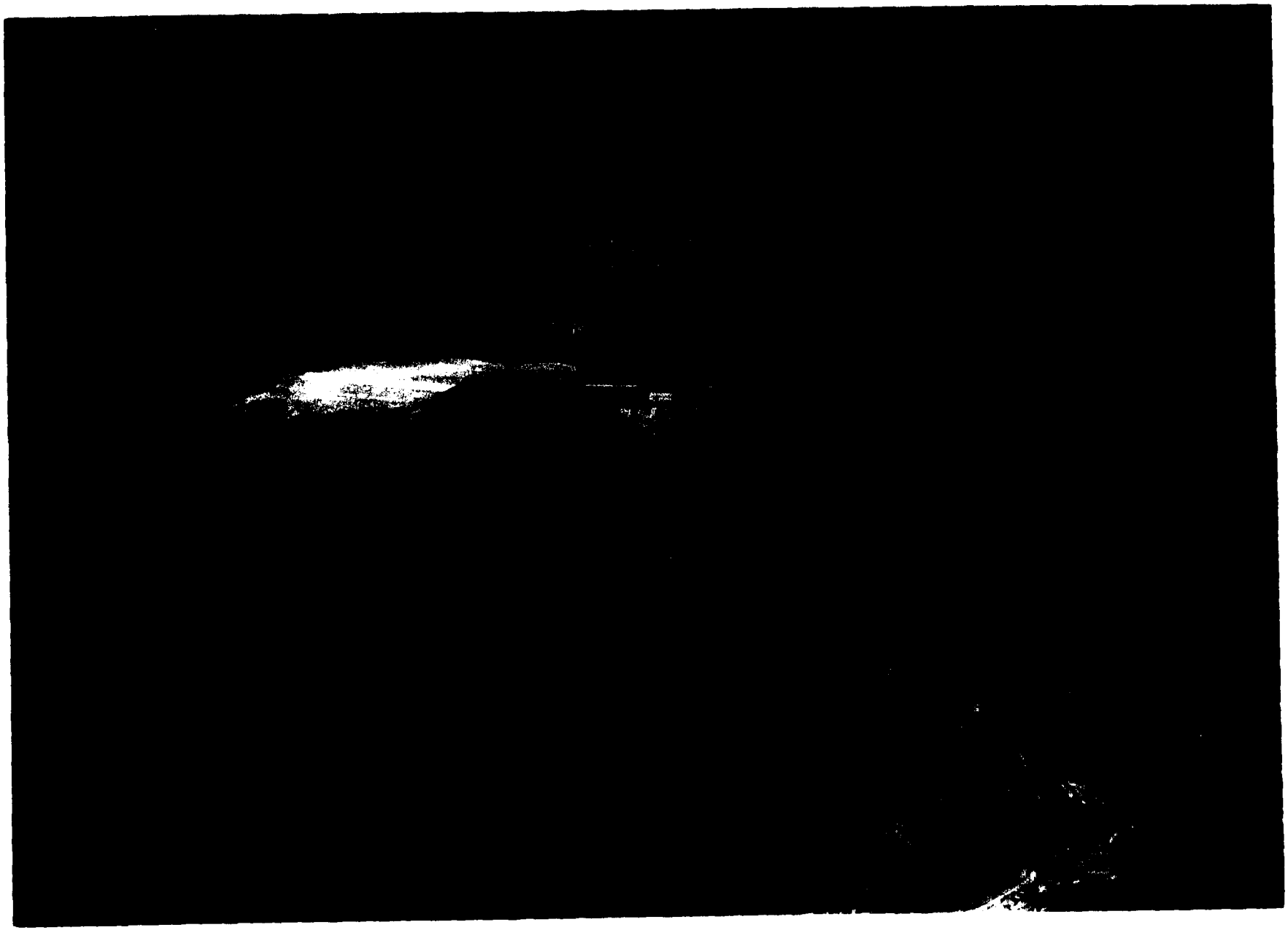
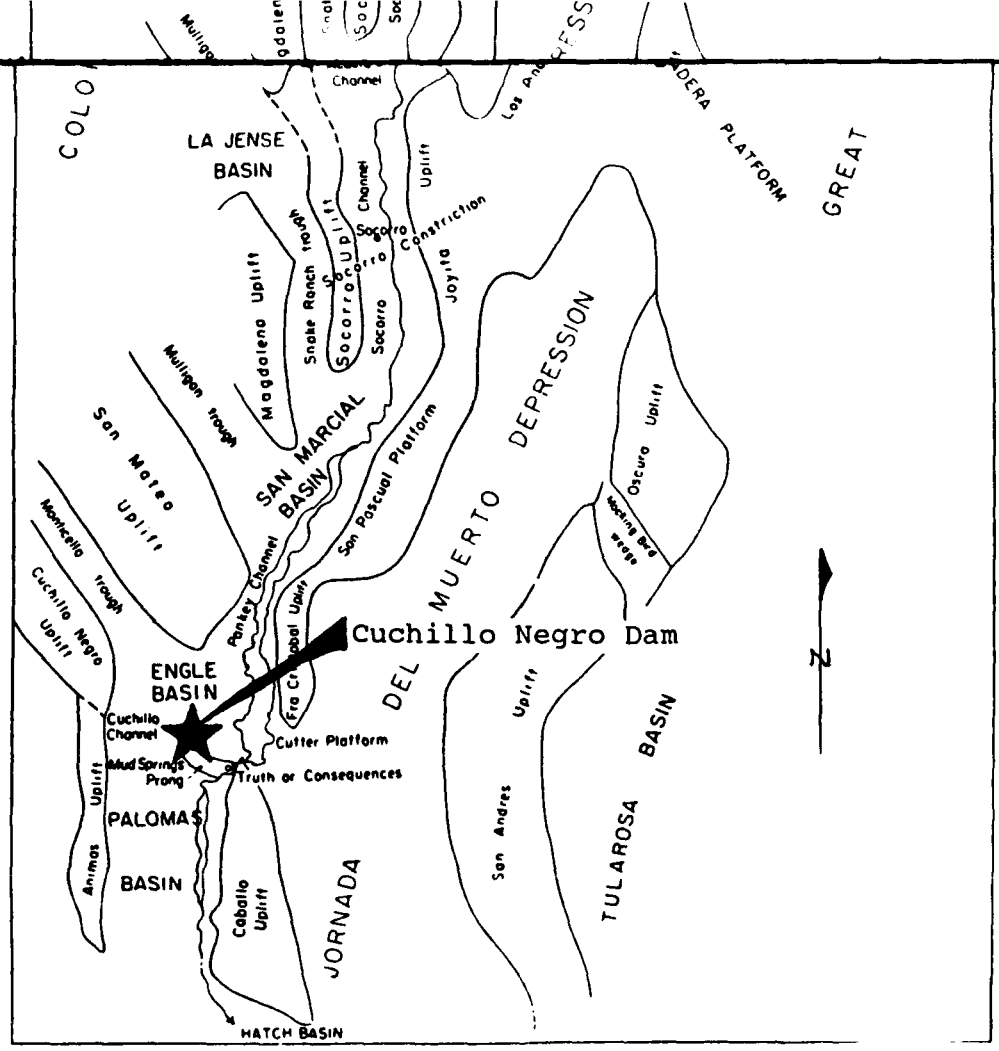


Plate No. A-3 Aerial View of Completed Project, Looking Upstream.

SAN LUIS BASIN
Taos
Picuris Prong
Cerro Negro
Embudo Constriction
Penasco Embayment
ESPAÑOLA BASIN
Española
Santa Fe
Santa Fe Embayment
Galisteo Platform
La Bajada Constriction
Cerrillos Uplift
Hagan Embayment
White Rock Channel
SANTO DOMINGO BASIN
Jemez
Caldera
Shield
Nacimiento Uplift
ALBUQUERQUE
Albuquerque
Sandia Uplift
Manzanita Uplift
Manzano Uplift
Los Pinos Uplift
BELEN BASIN
Belen
San Acacio Channel
Ladron Uplift
LA JENSE BASIN
Puerco Plateau
Lucero Uplift
COLORADO PLATEAU
GREAT PLAINS
CHUPADERA PLATFORM
ESTANCIA BASIN
Tectonic Diagram of Rio Grande Depression
Rio Grande depression
Uplifts with thin sed plugs
Trails or Consequences
Socorro



U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
Albuquerque, New Mexico

DES. BY CBD	RIO GRANDE & TRIBUTARIES — TRUTH OR CONSEQUENCES, N.M.
DWN. BY CBD	CUCHILLO NEGRO DAM Foundation Report
CHK. BY DEL	MAP OF A PART OF THE RIO GRANDE DEPRESSION SHOWING MAJOR TECTONIC FEATURES

David E. Wright
ENGINEER

DATE: 9-4-92

Plate No. A-4

DEPARTMENT OF THE INTERIOR
U.S. GEOLOGICAL SURVEY

GEOLOGIC MAP OF THE CUCHILLO QUADRANGLE, SIERRA COUNTY, NEW MEXICO

By Charles H. Maxwell and Mariel R. Oakman

Plate No. A-5
GEOLOGIC QUADRANGLE MAP
Published by the U.S. Geological Survey, 1990

MAXWELL AND OAKMAN—GEOLOGY, CUCHILLO QUAD., NEW MEXICO 1:24,000 MAP GQ-1686

GEOLOGIC MAP SYMBOLS

COMMONLY USED ON MAPS OF THE UNITED STATES GEOLOGICAL SURVEY

(Special symbols are shown in explanation)

Contact – Dashed where approximately located; short dashed where inferred; dotted where concealed

Contact – Showing dip; well exposed at triangle

Fault – Dashed where approximately located; short dashed where inferred; dotted where concealed

Fault, showing dip – Ball and bar on downthrown side

Normal fault – Hachured on downthrown side

Fault - Showing relative horizontal movement

Thrust fault – Sawteeth on upper plate

Anticline – Showing direction of plunge; dashed where approximately located; dotted where concealed

Asymmetric anticline – Short arrow indicates steeper limb

Overturned anticline – Showing direction of dip of limbs

Syncline – Showing direction of plunge; dashed where approximately located; dotted where concealed

Asymmetric syncline – Short arrow indicates steeper limb

Overturned syncline – Showing direction of dip of limbs

Monocline – Showing direction of plunge of axis

Minor anticline – Showing plunge of axis

Minor syncline – Showing plunge of axis

Strike and dip of beds – Ball indicates top of beds known from sedimentary structures

Inclined Horizontal

Vertical *Overturned*

Strike and dip of foliation

Inclined Vertical Horizontal

Strike and dip of cleavage

Inclined Vertical Horizontal

Bearing and plunge of lineation

Inclined Vertical Horizontal

Strike and dip of joints

Inclined Vertical Horizontal

Note: planar symbols (strike and dip of beds, foliation or schistosity, and cleavage) may be combined with linear symbols to record data observed at same locality by superimposed symbols at *point of observation*. Coexisting planar symbols are shown intersecting at point of observation.

Shafts

Vertical Inclined

Adit, tunnel, or slope

Accessible Inaccessible

Prospect

Quarry

Active Abandoned

Gravel pit

Active Abandoned

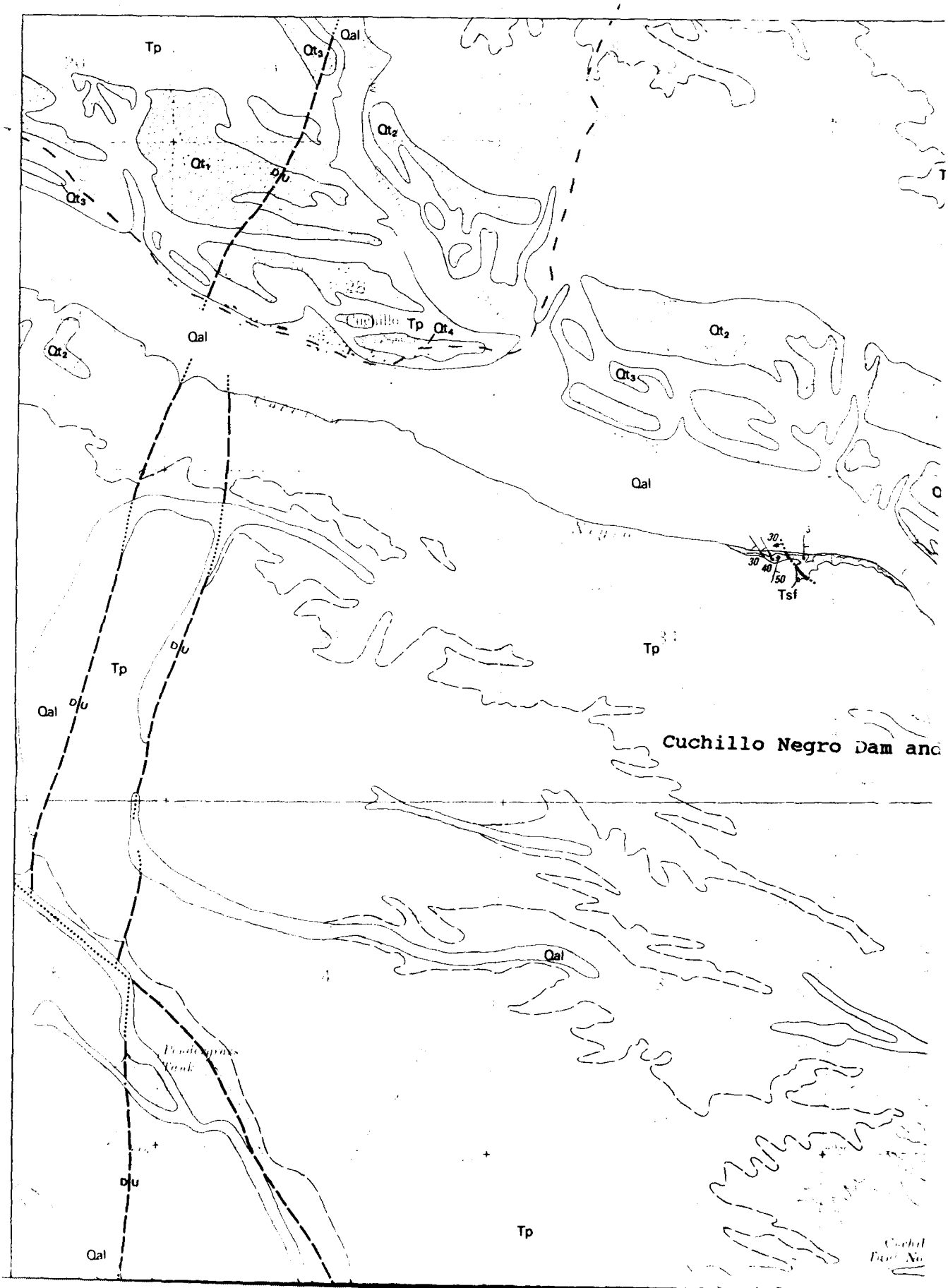
Oil well

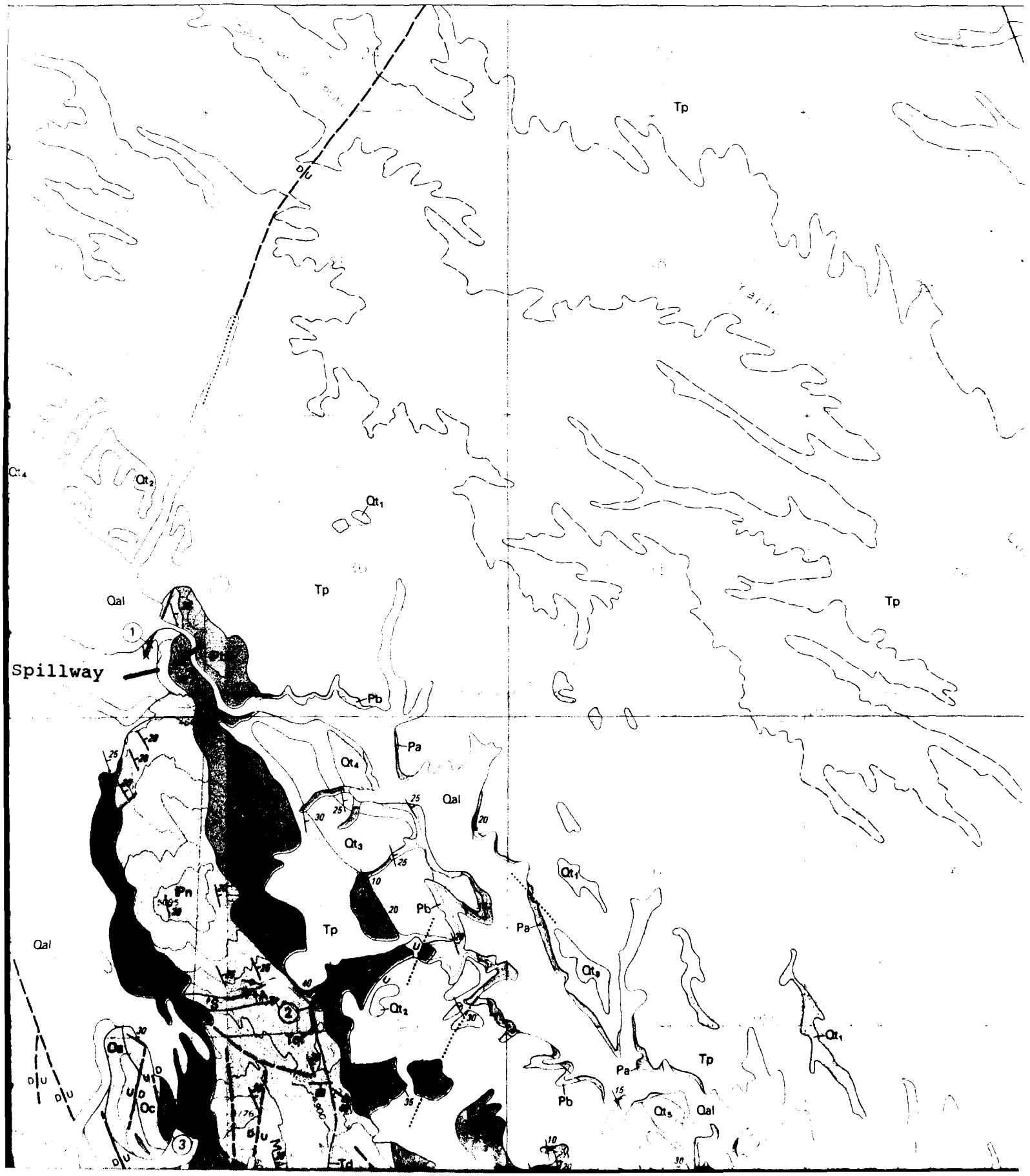
Drilling Shut-in Dry hole abandoned

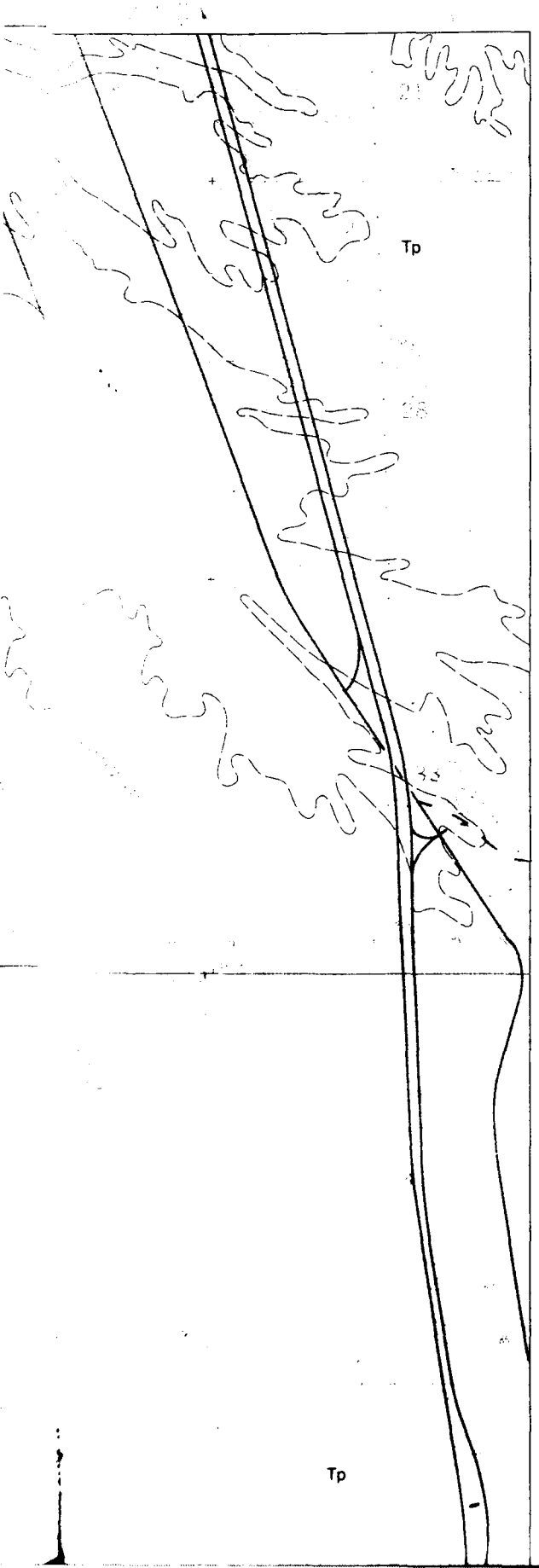
Gas Show of gas

Oil Show of oil

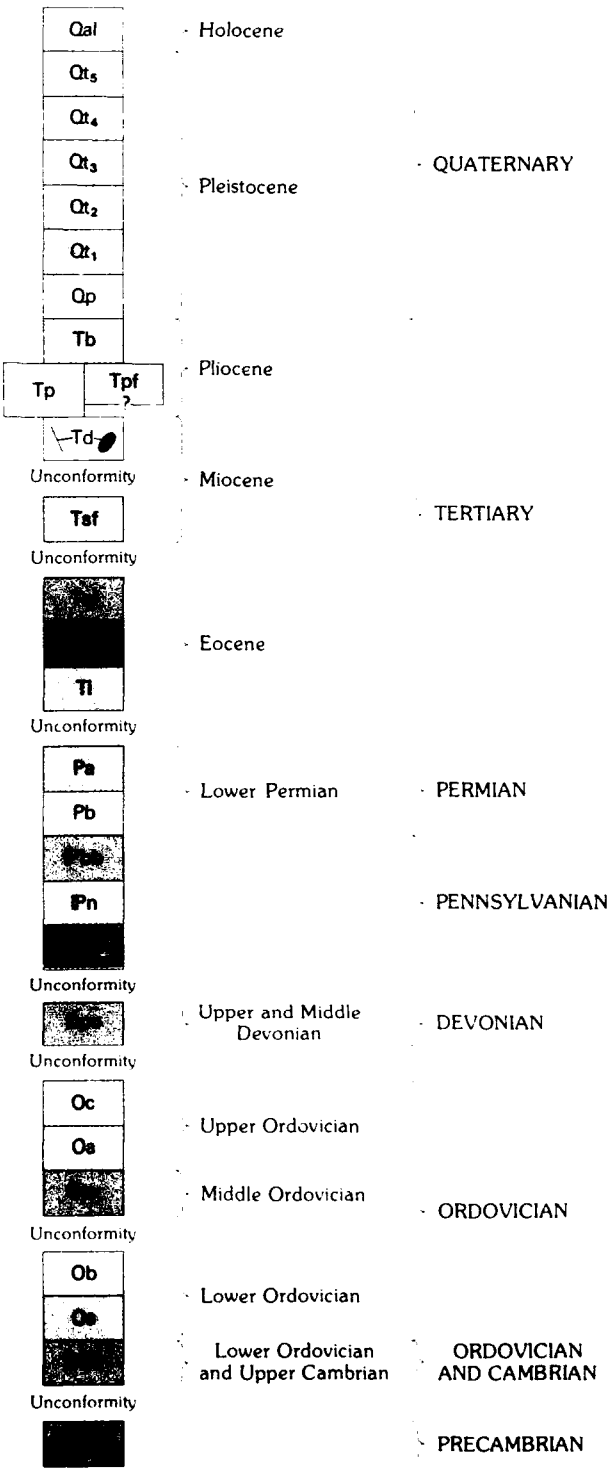
DEPARTMENT OF THE INTERIOR
U.S. GEOLOGICAL SURVEY




Tp
Qt4
Qt2
Qt1
Qal
Tp
Tp
Spillway
Pb
Pa
Qt4
Qal
Qt3
Qt1
Pn
Pb
Tp
Pa
Qal
Qt2
Qt3
Qt1
Pa
Tp
Pb
Qt5
Qal
Oc
Td



CORRELATION OF MAP UNITS



DESCRIPTION OF MAP UNITS

Qal **Alluvium (Holocene)**—Unconsolidated sand, silt, and gravel. Includes some alluvial fan deposits and remnants of terrace gravel deposits

Terrace gravel deposits (Pleistocene)—Four levels of terraces were developed on slopes of the incised valley of Cuchillo Negro Creek upstream of the box canyon (lat 33°13', long 107°19'), and five levels

Tl **Latite sill (Eocene)**—Sill intruding lower part of Pennsylvanian Red House Formation (IPr) in southern half of Mud Springs Mountains; exposed for about 2 mi along strike; 0–90 ft thick. Light greenish gray and light to dark olive green and brown; very fine grained, local medium-grained lenses; nesophitic texture, locally plumose, trachytic near margins. Feldspar laths, 0.02 by 0.2 to 0.2 by 1.0 mm, partly altered to clay; scattered orthoclase and rounded quartz grains 0.2–0.6 mm in diameter. Metasomatism partly to completely altered hornblende to actinolite; interstitial material altered to chlorite, which weathers to clay, limonite, and calcite; minor biotite and muscovite. Trace-element and major-oxide analyses in tables 1 and 2 (sample No. 8). Hornblende $^{40}Ar/^{39}Ar$ age of 43.4±0.2 Ma (L.W. Snee, written commun., 1989)

Pa **Abo Formation (Lower Permian)**—Dark-red shale and sandstone; minor lenses of orange-red arkosic sandstone and conglomerate and purplish-gray nodular calcareous mudstone. Grades into underlying Bursum Formation. About 500 ft exposed

Magdalena Group

Pb **Bursum Formation (Lower Permian)**—Moderate-red, green, and purplish-gray shale and calcareous shale; minor red sandstone and gray limestone. Nodular purplish-gray argillaceous limestone at base and top of unit. Transitional from underlying marine to overlying continental sedimentary rocks. About 300 ft thick

Pbb **Bar B Formation (Pennsylvanian)**—Light- to dark-gray, thin- to medium-bedded cherty limestone; chert commonly mottled and weathered to tan or brown. Alternates with dark-gray, thin- to thick-bedded shale and minor greenish-gray and reddish-gray siltstone and shale; shale predominant over limestone. Grades into overlying Bursum Formation and interfingers with underlying Nakaye Formation; lower contact drawn where shale is more abundant than limestone (Kelley and Silver, 1952). Thickness 500–700 ft

Pn **Nakaye Formation (Pennsylvanian)**—Medium- to dark-gray, very fine grained, thick- to massive-bedded cherty limestone with dark-gray thin shale interbeds; chert locally abundant in bands, lenses, and nodules. About 80 percent of unit is limestone (Kelley and Silver, 1952). Thickness 500–650 ft

Red House Formation (Pennsylvanian)—Light-gray and moderate-greenish-gray shale and calcareous shale containing limestone nodules and lenses; interbedded with light-gray limestone and calcarenite and a few thin sandstone lenses. Thin to thick beds of cherty limestone near top of unit; massive cherty limestone beds at base of unit in southern half of Mud Springs Mountains, but not present in northern half.

Overlies Upper Devonian Percha Shale (Dpo) in southern half of Mud Springs Mountains and overlies Upper Ordovician Cutter Dolomite (Oc) in northern half. Underlying Cutter Dolomite is generally concordant, locally discordant. This local unconformity appears to be within the lower part of the Pennsylvanian section (Maxwell and Oakman, 1986), between 100 and 200 ft above the base. Over 300 ft of sedimentary rocks missing from the northern half of the mountains are present in the southern half. The unconformity is marked by a layer a few inches to 3 ft thick composed of hard, rounded, irregular-shaped composite nodules of chert and limestone in a soft matrix of very fine grained calcite and clay, locally covered by a thin caliche layer. Lower surface of chert zone is irregular; upper surface is even, grading upward into about 15 in. of soft-weathering green shale containing numerous small chert pebbles that become smaller and sparser toward top of shale. An overlying calcarenite bed about 15 in. thick also has scattered chert pebbles in its basal layer.

The chert nodules are cryptocrystalline, with a few scattered quartz crystallites, small irregular lenses containing angular silt-size quartz, scattered rounded fine grains of detrital quartz and chert, and irregular lenses of fine-grained calcite. The chert in the nodules is isotropic, with local small areas that show vague aggregate polarization, and areas that contain a dusting of microlites that may be concentrated into indistinct concentric or parallel bands; local

vel Includes
avel deposits
erraces were
Negro Creek
nd five levels

MINES AND PR

Mines and prospects in the Cuchillo quac
indicate those that were sampled. Trace-eler
the samples and localities are listed in tabl
intrusive rocks are listed in table 2). The Equa
(sample Nos. 6–7), was reported to have p
(Harley, 1934, p. 194) from a vertical breccia
the breccia from the walls of the old wo
argentojarosite, plumbojarosite, and other s
dolomite and ankerite (sample No. 6). Samp
(sample No. 7, for example) contain numer
various shades of blue, green, brown, and
murdochite. Several prospects in the southern
barite, and one (sample No. 5) contains ba
prospects are on manganese-calcite veins. Th
Consequences produced about 17,000 tons
spring mineralization of the Palomas Form
minerals are pyrolusite and cryptomelane.

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Mineral Resources Circular 189, 32 p.

[Analyses by semiquantitative
spectrographic, N.M. Conklin
six-step series 10, 7, 5, 3,

ECT MINES AND PROSPECTS

e are the Cuchillo quadrangle are shown on the map; numbers
najor mpled. Trace-element analyses and short descriptions of
ne, r are listed in table 1 (major-oxide analyses of selected
ed at table 2). The Equator mine, now the Three Brothers mine
6–10 ported to have produced about $40,000 in horn silver
s ha n a vertical breccia pipe 6–10 ft in diameter. Fragments of
lary lls of the old workings have small clots of embolite,
om r osite, and other secondary minerals with iron oxides in
nide nple No. 6). Samples from mantos north of the old shaft
, and le) contain numerous unidentified secondary minerals in
of the reen, brown, and black, and tiny black octahedrons of
celest ects in the southern part of the map area have small veins of
1 195 No. 5) contains barite, celestite, and galena. Most other
v-gra se-calcite veins. Through 1957, the Ellis claims in Truth or
(Far bout 17,000 tons of low-grade manganese ore from hot-
he Palomas Formation (Farnham, 1961). Principal ore
d cryptomelane.

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rcular 189, 32 p.

Table 2.—*Major-oxide analyses of selected igneous rocks*

[Values in weight percent. Analyses by X-ray spectrography; analysts: A.J. Bartel and H.G. Neiman. LOI, loss on ignition]

Sample No. on map--	2	4	8
SiO_2	63.5	70.4	56.6
Al_2O_3	14.2	14.8	16.1
Fe_TO_3	4.26	1.73	4.81
MgO	1.73	1.01	3.10
CaO	2.39	0.51	4.78
Na_2O	3.93	1.26	4.81
K_2O	4.42	5.65	2.58
TiO_2	0.76	0.20	0.73
P_2O_5	0.33	0.13	0.29
MnO	0.06	0.03	0.06
LOI	2.66	2.96	4.18
F	0.03	0.02	0.02
Total	98.27	98.70	98.06
O=F	0.01	0.01	0.01
Total	98.26	98.69	98.05

SAMPLE DESCRIPTIONS

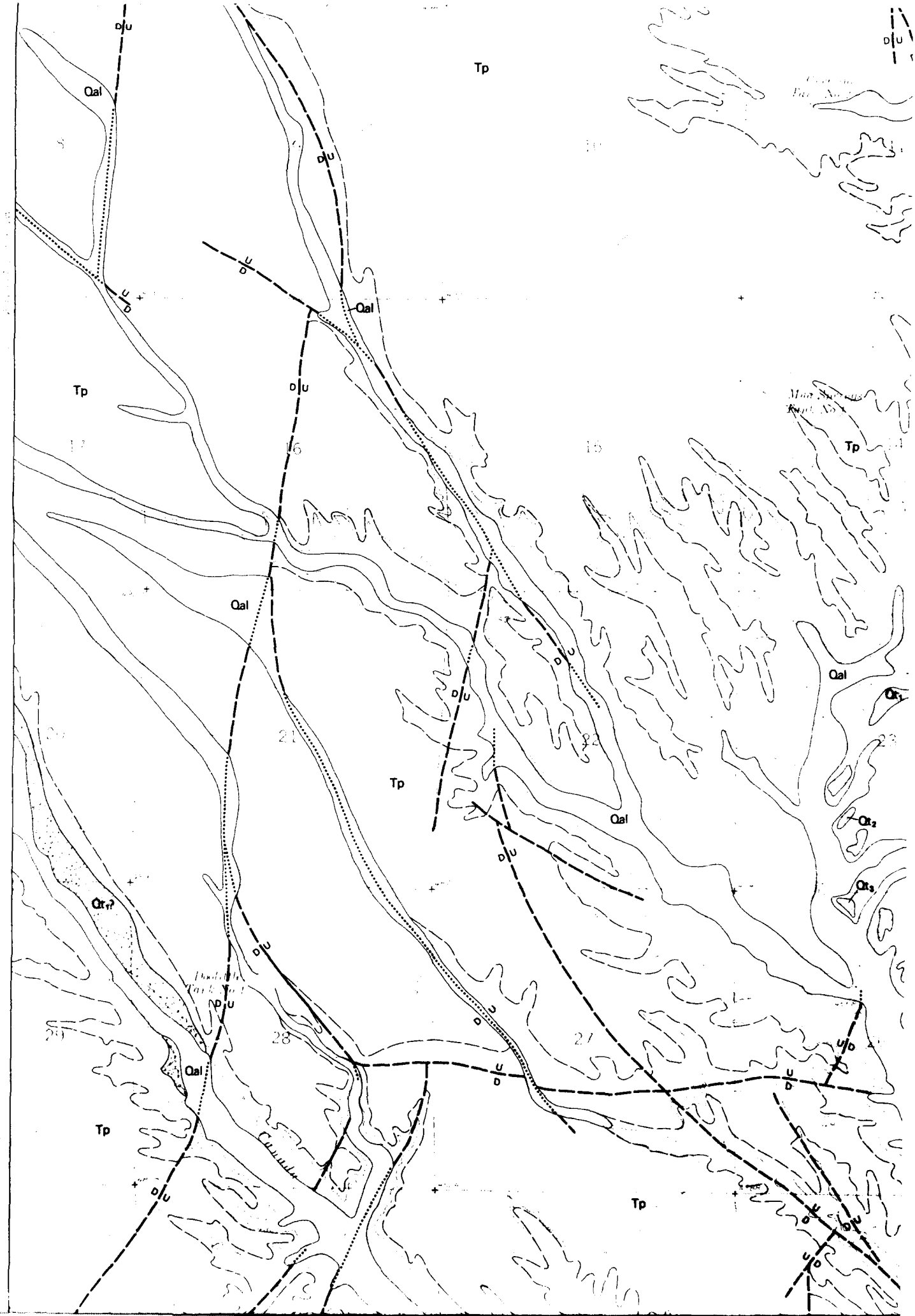
2. Porphyritic quartz latite dike (Tql).
4. Rhyolite dike (Tr) in Bliss Sandstone.
8. Latite sill (Tl) in Red House Formation.

CONVERSION FACTORS

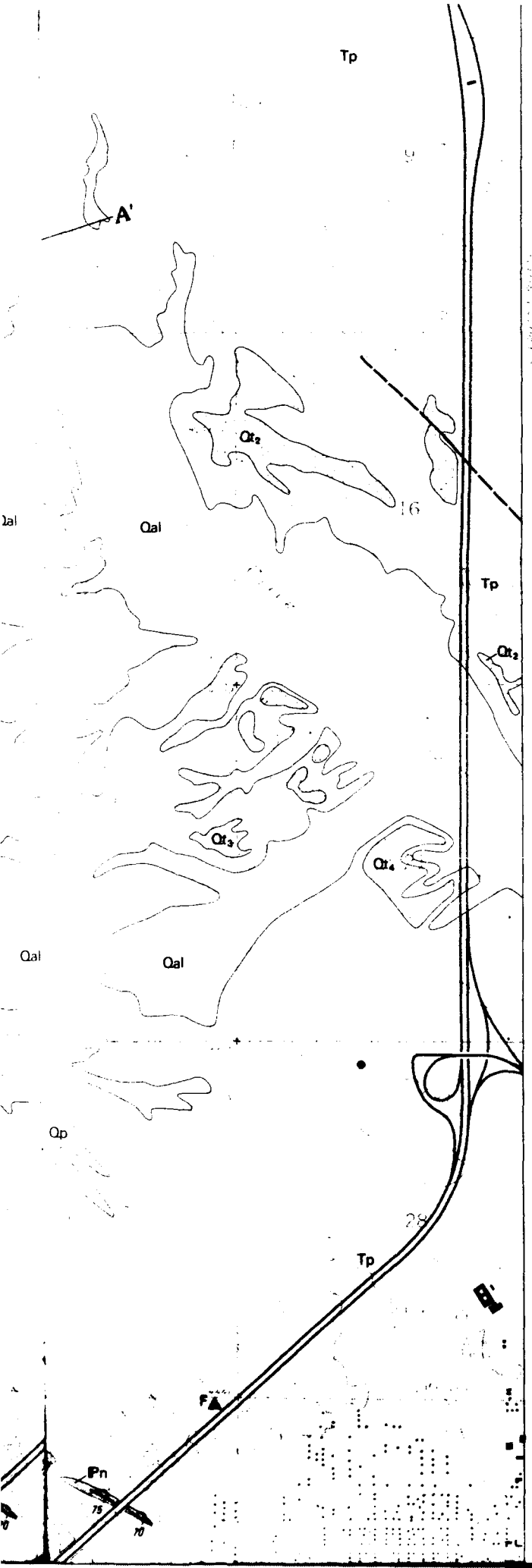
Multiply	By	To obtain
inches (in)	2.54	centimeters (cm)
feet (ft)	0.3048	meters (m)
miles (mi)	1.609	kilometers (km)

Table 1.—*Trace-element analyses of selected igneous, altered, and mineralized rocks*

1.— rogr L, .5, titative spectrography, except sample Nos. 11 and 13 by inductively coupled plasma. Analysts: .M. Conklin and L. Bradley; ICP, J.C. Crock. Spectrographic results reported to the nearest number in the ... N, not detected at ... at value shown)


Tp
Qal
Qtr1
Qtr2
Qtr3
Qtr1?
D
U


Tp
Pa
Pb
Qt5
Qal
Qt2
Qp
Qt4
Td
Qt3
Qt6
Tpf
Qsw
Qb
Qbl
Qc
OCb
A
B
B'
C
C'
pCr
Tr
Dpo
Qt1
Pbb
18
19
TRAIL
Mountain
Mud
Tank



DESCRIPTION OF MAP UNITS

Qal **Alluvium (Holocene)**—Unconsolidated sand, silt, and gravel In some alluvial fan deposits and remnants of terrace gravel de

Terrace gravel deposits (Pleistocene)—Four levels of terraces developed on slopes of the incised valley of Cuchillo Negro upstream of the box canyon (lat 33°13', long 107°19'), and five downstream (see fig. 1). Similar terraces, generally only four also developed on the incised valleys of the Rio Grande an Springs Canyon, as well as on all other major streams in the r West of Mud Springs Mountains on Mud Springs Canyon and C Negro Creek, as well as on other valleys in the region, terrac almost entirely on north slopes of valleys and are rarely preser south slopes. However, east of Mud Springs Mountains on C Negro Creek, most terraces are on south slopes.

Most terraces have thin veneers of sand and gravel or sc developed on valley floors during periods of low stream flow. Ir of the valleys to the next level occurred during periods of g increased flow, probably during intervals of pluvial climate asso with glacial stages. Highest terrace may be pre-Illinoian, one two lowest is Wisconsin (Hunt, 1978)

Qt5 **Fifth or youngest terrace**—10–40 ft above level of Cuchillo Creek. Silt, sand, and gravel; generally good soil development, eroded to bare rock outcrop

Qt4 **Fourth terrace**—20–50 ft above creek level. Silt, sand, and g generally good soil development

Qt3 **Third terrace**—50–100 ft above creek level. Thin soil on san gravel; minor caliche

Qt2 **Second terrace**—90–120 ft above creek level. Thin soil on e sand and gravel; poc.ly developed caliche

Qt1 **First or oldest terrace**—150–200 ft above creek level and 50– below Cuchillo pediment surface; largely eroded away or dissected except in northwest corner of quadrangle. Caliche is thick; little soil or detritus above caliche

Qp **Pediment deposits (Pleistocene)**—Present on east and south si Mud Springs Mountains. Sand and gravel similar to that of te but with angular limestone and chert clasts; generally well deve caliche, especially on southeast side of mountains. Appa related to oldest terraces, but difficult to correlate (see f Generally thin, 1–3 ft, locally as much as 20 ft

Tb **Basalt (Pliocene?)**—Olivine basalt flow and scoria; one small area 1 mi southeast of Mud Mountain. Similar in composition elevation to numerous tholeiitic basalt flows elsewhere in which were dated at 2.1–2.9 m.y. (Bachman and Mehnert, 1

Tp **Palomas Formation (Pliocene)**—Light- to medium-gray, len gravel and sandy gravel interbedded with light-pink, tar greenish-gray silt, sand, and conglomeratic sand, white san moderate-orange-red to brick-red and moderate-green mud Several altered ash beds near top of unit. Originally named Pa Gravels; renamed and subdivided into axial and piedmont faci considered as "upper Santa Fe Group" by Lozinski (1985 correlated with the Pliocene and Pleistocene Sierra La Formation to the north and the Pliocene and Pleistocene Cam Formation to the south. Not assigned to Santa Fe on this Overlies Santa Fe Formation with angular unconformity. Vert fossils from roadcut northwest of Truth or Consequences (vertebrate locality M-1481) indicate an age between 4.05 and 4.2 for lower part of Palomas (Repenning and May, 1986).

Extensive Cuchillo pediment surface well developed on Palomas Formation. Surface is remarkably even, with a con slope of about 50 ft/mi throughout the lower Rio Grande steepening to about 80 ft/mi near bordering mountains. Surfa local well-developed soil on silt, sand, and gravel pediment de and local disintegrating caliche deposits. Surface is eroded by tributaries and incised by major streams to relatively cor depths of 260–280 ft. Minimum age of surface is probably near 2 basalt flows interbedded with upper part of valley fill north and quadrangle were dated at 2.9 m.y., and flows on surface at 2.1–2 (Bachman and Mehnert, 1978)

Tpf **Fanglomerate of Mud Mountain (Pliocene)**—Light-gray fanglo and conglomerate composed predominantly of boulders, c and pebbles of limestone in a matrix of sandy marl; a fa Palomas Formation mapped separately on west side of Mud Mountains because of its distinctive color and lithology

Td **Basalt dikes (Miocene?)**—Basalt dike intruding Lower Permi Formation (Pa) (see section A–A') is 1–4 ft wide, vertical, da green and brown; nesophitic texture; highly altered with chal

quartz crystallites, small irregular lenses containing angular silt-size quartz, scattered rounded fine grains of detrital quartz and chert, and irregular lenses of fine-grained calcite. The chert in the nodules is isotropic, with local small areas that show vague aggregate polarization, and areas that contain a dusting of microlites that may be concentrated into indistinct concentric or parallel bands; local crosscutting dendritic growths of calcite show optical continuity

Percha Shale and Sly Gap and Oñate Formations, undivided (Upper and Middle Devonian)—Upper part generally dark-gray, fissile claystone and siltstone, locally calcareous; equivalent to Upper Devonian Percha Shale. Grades downward into light-gray to buff, tan-weathering calcareous siltstone; equivalent to Upper Devonian Sly Gap Formation (Sorauf, 1984). Local thin limestone layers and nodules, massive red-brown-weathering calcareous siltstone, and dolomite at base; equivalent to Middle Devonian Onate Formation. Thickness 10–100 ft

Montoya Group

Oc

***Cutter Dolomite* (Upper Ordovician)**—Light-gray- or light-tan-weathering dolomite, limestone, and siltstone; generally unfossiliferous. Dolomite generally thick bedded, light to dark gray, sublithographic; nodular chert in middle and upper part. Interbedded limestone is dark gray, thin bedded, microgranular. Basal part is calcareous siltstone and shale. Thickness 120–350 ft

Oa

Aleman Formation (Upper Ordovician)—Dolomite and chert in alternating layers. Dolomite is medium to coarse grained in layers 2–12 in. thick; weathers gray to brownish gray and variegated pink and white. Chert is white, brown, and black in irregular bands 1–3 in. thick; some nodular chert in upper part. Thickness 160–300 ft

Second Value Dolomite (Middle Ordovician)—Comprises two members (Hayes, 1975). Thickness 60–70 ft

Upham Dolomite Member—Dark-gray, brownish-gray-weathering, fine-grained to coarsely crystalline dolomite; contains minor irregular chert nodules, scattered detrital quartz grains, and irregular lenses of sandstone in basal part. Grades into underlying member

Cable Canyon Sandstone Member—Dark-gray-brown-weathering, locally crossbedded, coarse-grained granulitic sandstone and granule conglomerate

El Paso Group (Lower Ordovician)

Ob

Bat Cave Formation—Upper part is alternating medium- and dark-gray, very fine grained, medium- to thick-bedded, generally cliff forming limestone, dolomitic limestone, and minor dolomite; light- to dark-brown-weathering chert in bands and nodules; light-gray-weathering stromatolitic zones; numerous solution caverns and collapse breccia; equivalent to lower part of McKelligon Limestone (Hayes, 1975), upper part of McKelligon is not present in map area. Lenticular sandstone and conglomerate locally present near middle of formation; equivalent to upper sandy member of Hitt Canyon Formation (Hayes, 1975). Lower part is light- to moderate-gray, thin- to medium-bedded, generally slope forming limestone; many stromatolitic biostromes interspersed with detrital limestone; local nodular chert, chert bands, and laminae; equivalent to upper part of middle member of Hitt Canyon Formation (Hayes, 1975). Thickness 250–400 ft

Sierrite Limestone—Medium to dark gray with brownish cast, fine to very fine grained, thin to medium bedded. Alternating limestone layers and thin undulating laminae of brown-weathering chert result in a distinctive wavy banded appearance; chert is light gray to white on fresh surfaces. Forms cliffs and steep slopes; weathers to thin brown slabs. Equivalent to lower part of Hitt Canyon Formation (Hayes, 1975). Thickness 150–200 ft

Bliss Sandstone (Lower Ordovician and Upper Cambrian)—Upper few feet of unit is transitional with overlying Sierrite limestone (Os); composed of thin-bedded, brown-weathering, laminated cherty limestone alternating with thin-bedded glauconitic sandy shale. Most of unit is alternating thin beds of gray glauconitic limestone and dolomite and fine-grained ferruginous or glauconitic sandstone and siltstone. Oolitic hematite bed 5–6 ft thick about 10–15 ft above base. Basal part is medium- to coarse-grained sandstone and conglomerate; base is locally a conglomerate of angular fragments of Precambrian rocks cemented by travertine. Thickness about 140 ft

Precambrian rocks, undivided—Complexly contorted and interlayered reddish-gray quartzite, dark-gray and brown quartz schist, quartz-biotite schist, amphibolite gneiss and schist, and intercalated red porphyritic granite gneiss

[Analyses by semiquantitative spectrographic, N.M. Conki six-step series 10, 7, 5, 3 leaders (--), not looked fo

Sample No. on map--	1	2	
Fe	0.3	3	
Mg	0.15	1.5	
Ca	3	3	>10
Ti	0.03	0.5	
Si	7	>10	7
Al	0.5	10	
Na	0.3	3	
K	0.7	3	<0
Mn	7	0.07	3
Ba	3	0.15	3
Ag	N5	N	N
Be	15	3	7
Co	30	20	15
Cr	15	50	15
Cu	70	30	20
La	30	150	30
Mo	70	<3	15
Nb	30	20	20
Ni	30	30	7
Pb	<50	30	150
Sc	N5	15	N
Sn	N10	N	N
Sr	1500	1500	1500
V	300	150	30
W	3000	N100	3000
Y	15	70	30
Zn	N300	N	
Zr	N10	200	
Ce	N	200	
Ga	70	30	N
Ge	30	N10	
Tl	70	N50	
Yb	2	7	
Nd	N70	100	
As	N1000	N	
Th	--	--	

1. Manganese prospect in fa (Pn).
2. Porphyritic quartz latit
3. Manganese-calcite vein i
4. Rhyolite dike (Tr) in Bl
5. Barite-galena vein in fo
6. Equator/Three Brothers m
7. Equator/Three Brothers m

Table 1.—*Trace-element analyses of selected igneous, altered, and mineralized rocks*

tative spectrography, except sample Nos. 11 and 13 by inductively coupled plasma. Analysts: Conklin and L. Bradley; ICP, J.C. Crock. Spectrographic results reported to the nearest number in the 7, 5, 3, 2, 1.5, 1, etc. H, interference; N, not detected at limit of detection or at value shown; ked for]

	3	4	5	6	7	8	9	10	11	12	13	14
					IN WEIGHT PERCENT							
	0.15	1.5	0.7	10	7	5	1.5	3	0.06	3	0.37	5
5	0.15	1	0.1	0.7	0.15	2	1.5	0.1	0.64	0.5	0.1	0.1
	>10	0.3	0.7	>10	1	3	>10	3	0.65	0.7	0.57	1.5
.5	0.007	0.15	0.007	0.015	0.03	0.3	0.015	0.007	<0.04	0.15	<0.04	0.007
	7	>10	3	1	7	>10	1.5	>10	--	10	--	>10
	0.2	7	0.15	0.5	0.7	10	0.5	0.3	0.14	7	0.53	0.15
	0.05	1	<0.05	0.07	H	5	0.05	0.07	--	1.5	--	<0.05
	<0.7	5	<0.7	0.7	<0.7	3	<0.7	<0.07	0.5	3	<0.4	<0.7
.07	3	0.05	0.03	1.5	1.5	0.07	1.5	3	>38	1.5	>45	1.5
.15	3	0.15	>10	0.05	0.03	0.2	0.5	1.5	9.4	0.3	0.83	0.5
					IN PARTS PER MILLION (Except where noted in weight percent)							
	N	N	7	150	150	N	N	N	<20	N	<20	N
	7	2	<1	<1	3	<1	7	15	47	10	42	30
	15	7	<5	7	150	30	<5	10	600	15	30	15
	15	10	3	7	5	300	7	5	18	70	12	3
	20	1	15	150	>10%	50	7	30	2400	30	470	30
	30	30	<50	<30	<30	70	<50	<50	140	30	30	<50
	15	<3	30	15	10	<3	7	30	140	10	110	10
	20	15	<10	<10	<10	20	<10	10	<30	<10	<40	<10
	7	7	7	20	50	150	7	7	430	10	60	30
	150	30	5%	1500	7000	<10	150	100	620	30	160	10
	N	N	N	N	5	15	N	N	<20	7	<20	N
	N	N	N	70	70	10	N	N	<80	N	<90	N
	1500	30	3%	150	15	2000	1000	700	2700	700	570	200
	30	10	15	70	70	200	100	30	450	150	240	70
	3000	N	N	N	N	N	N	700	--	300	--	200
	30	15	N10	10	20	20	20	20	80	15	70	15
	N	N	N	700	1.5%	N	N	N	1500	N	140	N
	N	150	N	15	15	150	N	N	--	100	--	N
	H	N	N	N	N	150	H	N	--	N	--	N
	N5	30	N	H	H	30	30	15	70	30	<40	7
	N	N	N	N	N	N	N	15	--	N	--	30
	N	N	N	N	N	N	70	70	--	N	--	N
	3	1.5	<1	H	H	H	2	2	--	2	--	2
	N	N	--	--	--	70	--	--	60	N	<40	--
	N	N	N	N	N	N	N	N	200	N	<90	N
	--	--	--	--	--	--	--	--	160	--	180	--

SAMPLE DESCRIPTIONS

t in fault-vein in Nakaye Formation

z latite dike (Tql).

vein in jasperoid.

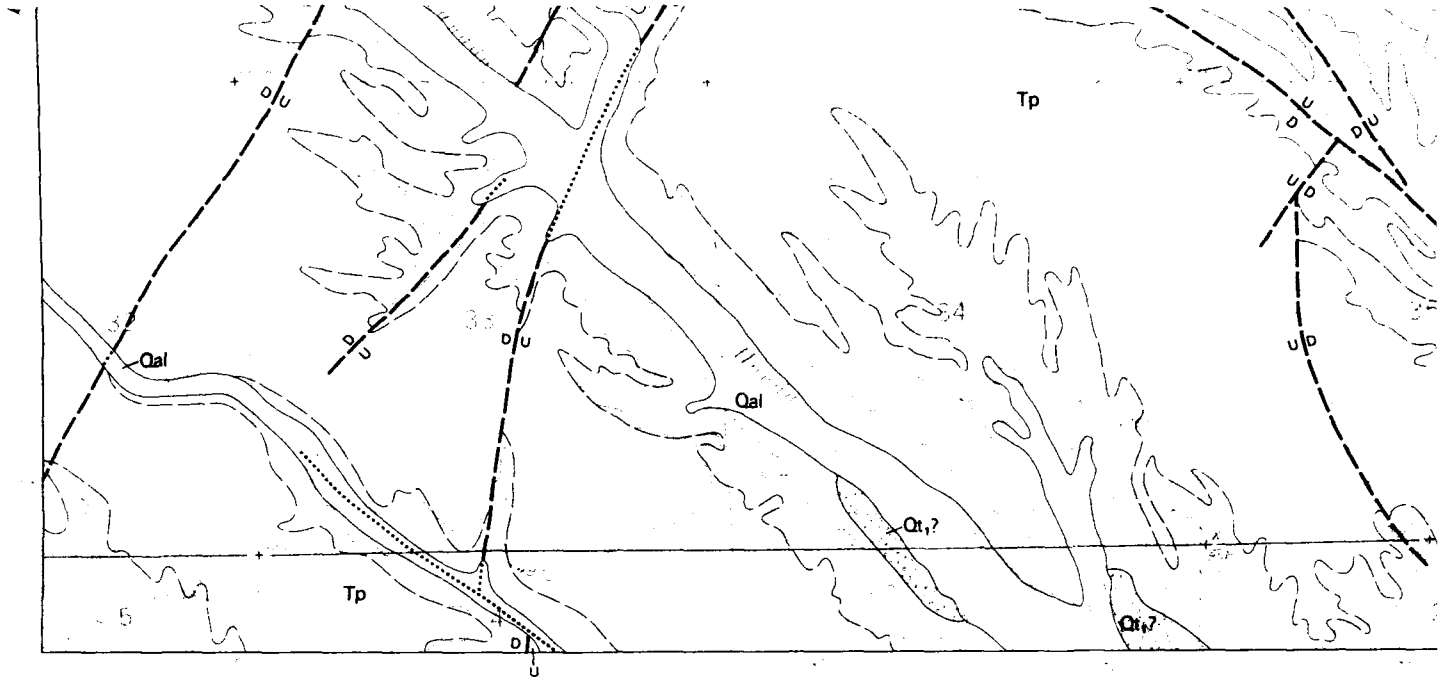
) in Bliss Sandstone.

n in footwall of fault.

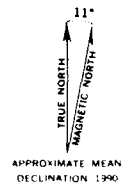
thers mine--wall of breccia pipe.

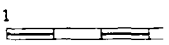
thers mine--copper manto ore.

8. Latite sill (Tl) in Red House Formation.
9. Calcite-manganese-jasperoid vein.
10. Jasperoid-manganese-calcite vein.
11. Ellis claims--manganese vein in altered dolomite.
12. Ellis claims--altered Palomas Formation (Tp) below manganese.
13. Pyrolucite ore overlying sample No. 12.
14. Jasperoid-manganese vein in overturned Red House Formation (Pr).

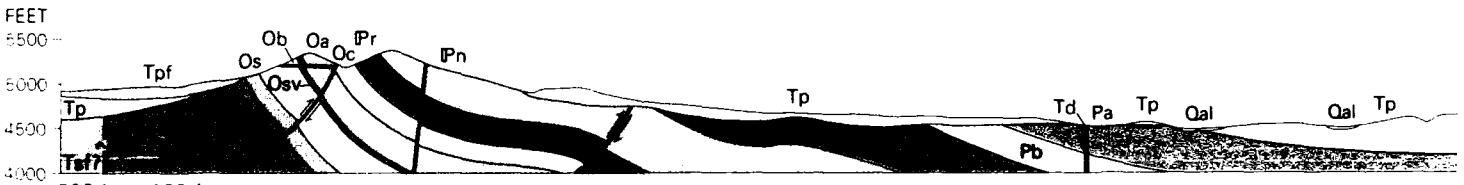


Base from U.S. Geological Survey, 1961
Photorevision as of 1980
10,000-foot grid based on New Mexico coordinate system, west zone
1000-meter Universal Transverse Mercator *grid ticks, zone 13, shown in blue*

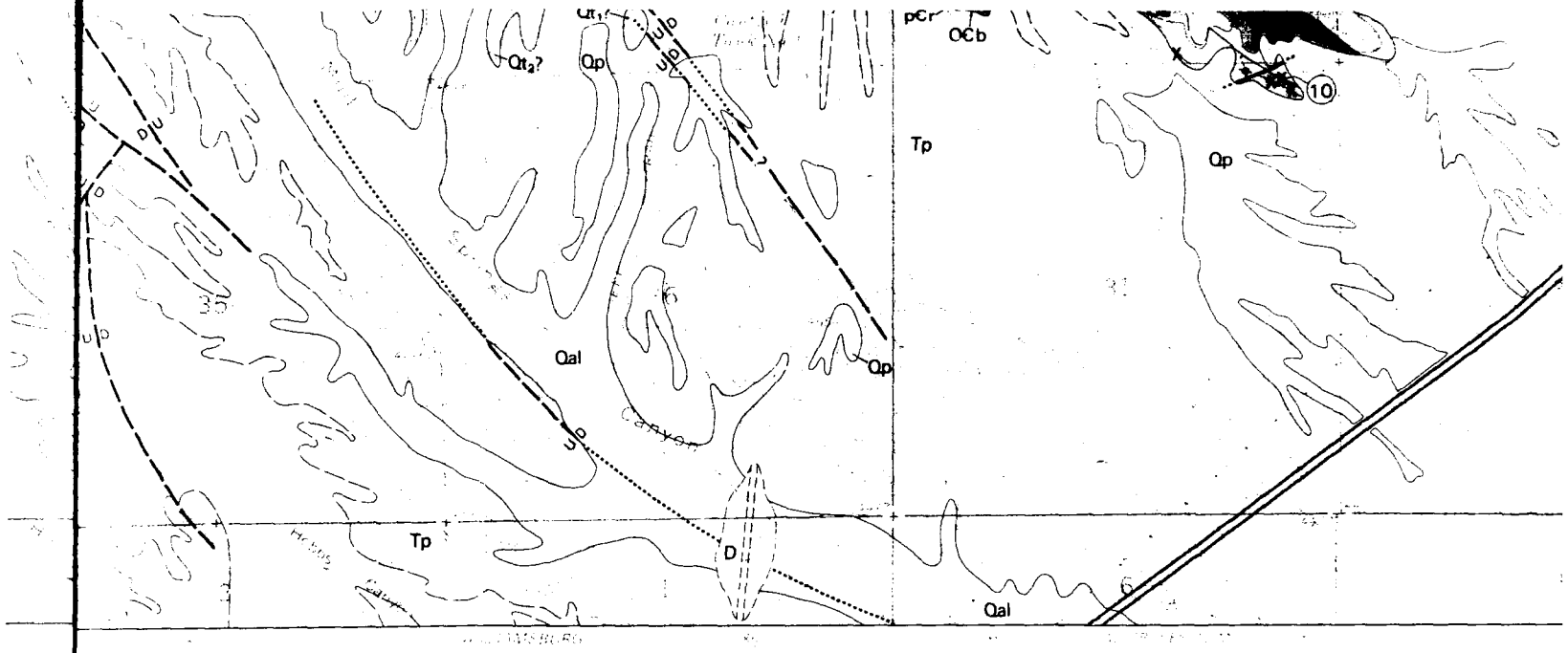




A



500 feet = 152.4 meters
Selected Quaternary units omitted



SCALE 1:24 000

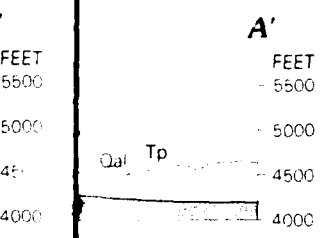
½ 0 1 MILE

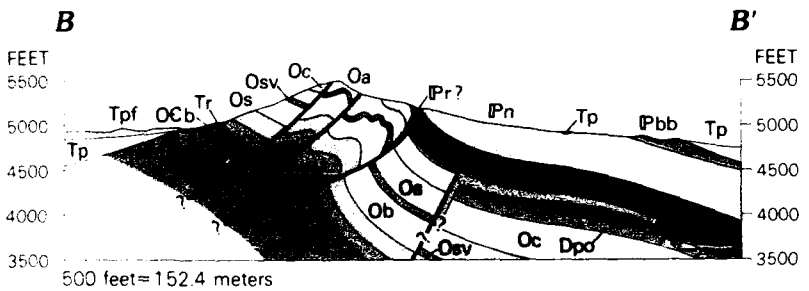
1 .5 0 1 KILOMETER

CONTOUR INTERVAL 20 FEET
NATIONAL GEODETIC VERTICAL DATUM OF 1929

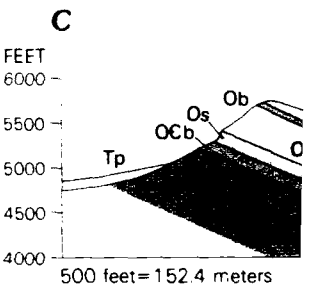


QUADRANGLE LOCATION



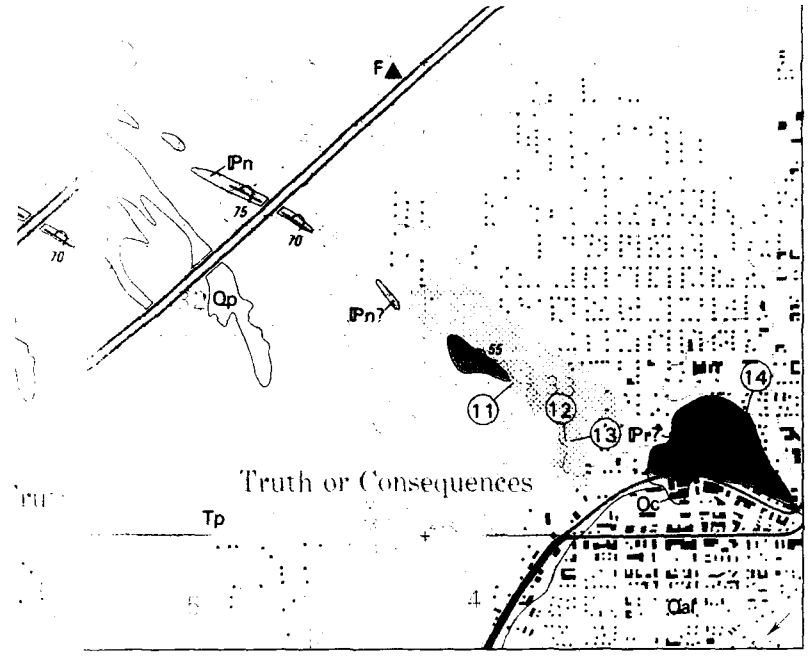


500 feet = 152.4 meters



500 feet = 152.4 meters

GEOLOGIC MAP

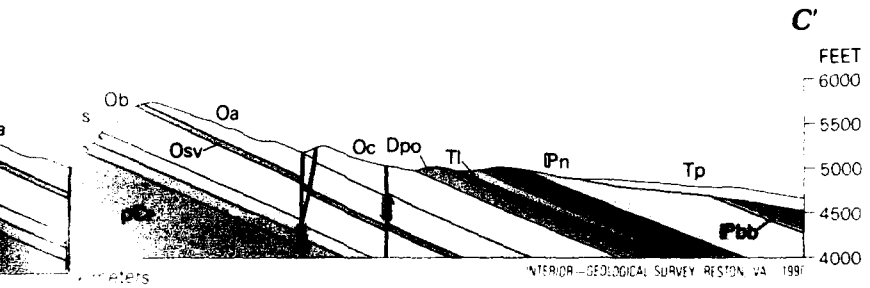


Geology mapped in 1981-82

Manuscript approved for publication, September 26, 1989

MEXICO

LOCATION



INTERIOR—GEOLOGICAL SURVEY, RESTON, VA 1990

Tpf **Fanglomerate of Mud Mountain (Pliocene)**—Light-
and conglomerate composed predominantly of
and pebbles of limestone in a matrix of sandy
Palomas Formation mapped separately on west si
Mountains because of its distinctive color and lith

Td **Basalt dikes (Miocene?)**—Basalt dike intruding Lo
Formation (Pa) (see section *A–A'*) is 1–4 ft wide,
green and brown; nesophitic texture; highly altere
and chlorite in veinlets and in cavities, calcite inte
laths, and augite(?) altered to clay, limonite, and
laths generally about 0.02 by 0.2 mm; trunca
Palomas Formation. Basalt dike intruding Penns
(PPn) and Bar B (PPbb) Formations, adjacent to porp
dike (Tql), is 0.5–3.0 ft wide, dark brown; simil
intruding Pa but is more altered and has more in

Tsf **Santa Fe Formation, undivided (Miocene)**—Reddish-
and tan, in part tuffaceous, slightly indurated c
sandy mudstone, calcareous mudstone, and sa
folded, and tilted to dips of 30°–50°. Exposed only
Creek 1.5 mi southeast of Cuchillo

Porphyritic quartz latite (Eocene?)—Dike intrudi
Nakaye Formation (PPn); 4–18 ft wide, light to m
grained; bleached and altered zone as wide as 20
dike. White rounded quartz phenocrysts 1–2.5 mm
with opaque dark-brown reaction rims, and white
phenocrysts 7–15 mm in diameter. Feldspar p
altered to clay and calcite; larger grains have myr
granophyric rims; outermost rim poikilitic with in
and randomly oriented euhedral orthoclase and pl
Groundmass is composed of equant grains of ortho
plagioclase; random biotite and opaques, and
Trace-element and major-oxide analyses in tables
No. 2)

Porphyritic rhyolite (Eocene)—Dike intruding Lowe
Upper Cambrian Bliss Sandstone (OCb); pinkis
grained groundmass of feldspar and quartz;
phenocrysts 4–5 mm in diameter, relatively un
shaped, pink and white phenocrysts altered to cl
equant grains generally less than 0.01 mm in
element and major-oxide analyses in tables 1 and
K-Ar age of biotite is 40.8±1.5 Ma (R.F. Marvin,
1987)

F MAP OF THE CUCHILLO QUADRANGLE, SIERRA CO

By

Charles H. Maxwell and Mariel R. Oakman

1990

1-2 ?s on surface at 2.1–2.2 m.y.

iglom—Light-gray fanglomerate
s, cob antly of boulders, cobbles,
a facie sandy marl; a facies of
ud Sp n west side of Mud Springs
r and lithology
rmian ding Lower Permian Abo
, dark ft wide, vertical, dark olive
chalce nly altered with chalcedony
to felc alcite interstitial to feldspar
e; felc nite, and chlorite; feldspar
over n, truncated by overlying
an Na ng Pennsylvanian Nakaye
quartz it to porphyritic quartz latite
basalt wn; similar to basalt dike
al calci more interstitial calcite
pale-br Reddish-gray, pale-brown,
neratic urated conglomeratic and
e. Fau and sandstone. Faulted,
chillo N sed only in Cuchillo Negro

nnsylv intruding Pennsylvanian
n gray ight to medium gray, fine
oth sic ie as 20 ft on both sides of
meter. -2.5 mm in diameter, some
ink fel nd white to pink feldspar
crysts dspar phenocrysts partly
ic core ave myrmekitic cores and
ns of b with inclusions of biotite
lase cr se and plagioclase crystals.
, quartz s of orthoclase, quartz, and
titial c s, and interstitial calcite.
d 2 (sa n tables 1 and 2 (sample

lovician g Lower Ordovician and
y, very pinkish-gray, very fine
edral b quartz; euhedral biotite
d; irre ely unaltered; irregular-
d felds ed to clay and feldspar in
meter. mm in diameter. Trace-
mple N s 1 and 2 (sample No. 4).
en com Marvin, written commun.,

dolomite and fine-grained ferruginous or glauconitic sandstone and siltstone. Oolitic hematite bed 5–6 ft thick about 10–15 ft above base. Basal part is medium- to coarse-grained sandstone and conglomerate; base is locally a conglomerate of angular fragments of Precambrian rocks cemented by travertine. Thickness about 140 ft

Precambrian rocks, undivided—Complexly contorted and interlayered reddish-gray quartzite, dark-gray and brown quartz schist, quartz-biotite schist, amphibolite gneiss and schist, and intercalated red porphyritic granite gneiss

Contact—Long dashed where approximately located; dotted where *concealed*

Approximate boundary of Cuchillo pediment surface

Fault—Showing dip where measured. Dashed where approximately located or inferred; dotted where concealed. U, upthrown side; D, downthrown side. Arrows indicate relative lateral motion

Anticline

Syncline

Monocline—Showing trace and plunge of axis. A, anticlinal bend; S, synclinal bend

Drag fold—Showing plunge

Strike and dip of beds—Dot marks point of observation

Inclined

Vertical

Overturned

Jasperoid

Vein—Showing dip where measured. Dashed where inferred

Manganese mineralization

Fossil locality—*USGS vertebrate* locality M-1481

Quarry

Mine or prospect

Sample locality—See tables 1 and 2

Flood control dam—Approximately located

4. Rhyolite dike (Tr
5. Barite-galena vei
6. Equator/Three Bro
7. Equator/Three Bro

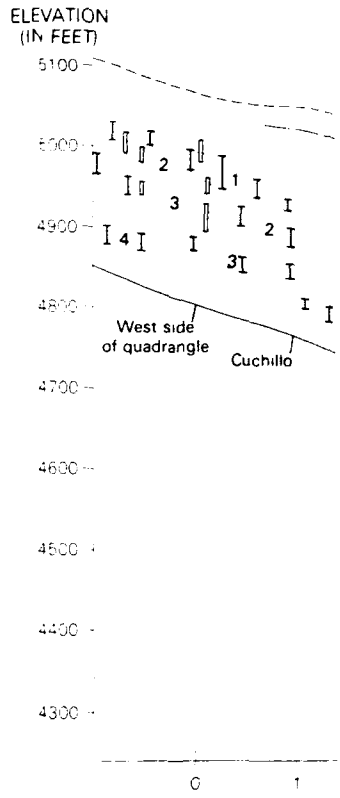
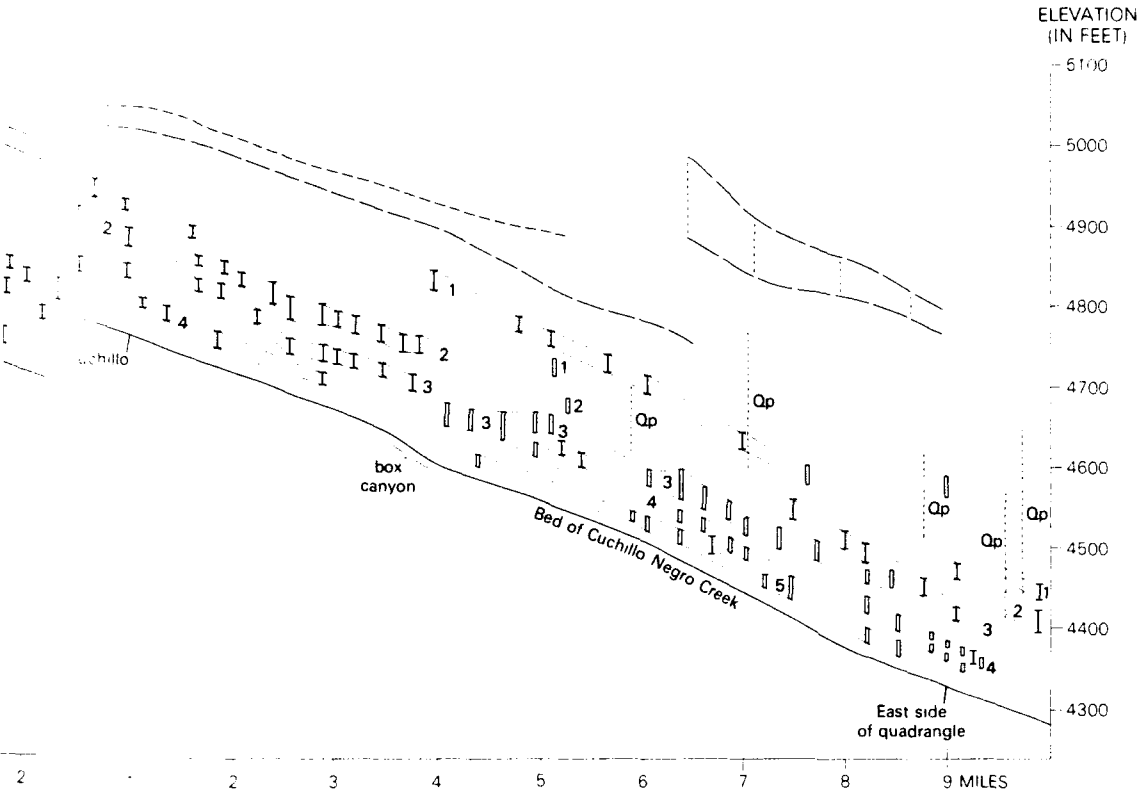


Figure 1.—Diagram show

IN COUNTY, NEW MEXICO

Bl[illegible]ke (Tr) in Bliss Sandstone.
fo[illegible]na vein in footwall of fault.
S[illegible]e Brothers mine--wall of breccia pipe.
S[illegible]e Brothers mine--copper manto ore.

11. Ellis claims--manganese vein in altered dolomite.
12. Ellis claims--altered Paloma Formation (Tp) below manganese.
13. Pyrolucite ore overlying sample No. 12.
14. Jasperoid-manganese vein in overturned Red House Formation (Pr).



EXPLANATION

Elevations and slopes of Cuchill pediment surface

North of Cuchillo Negro Creel

South of Cuchillo Negro Creel

East flank of Mud Springs Mountains

Qp Elevations of Qp pediment surfac south of Cuchillo Negro Cree on east flank of Mud Spring Mountains

Elevations and slopes of Pleist cene terraces—Numbers 1- correspond to map units Qt_1 Qt_5

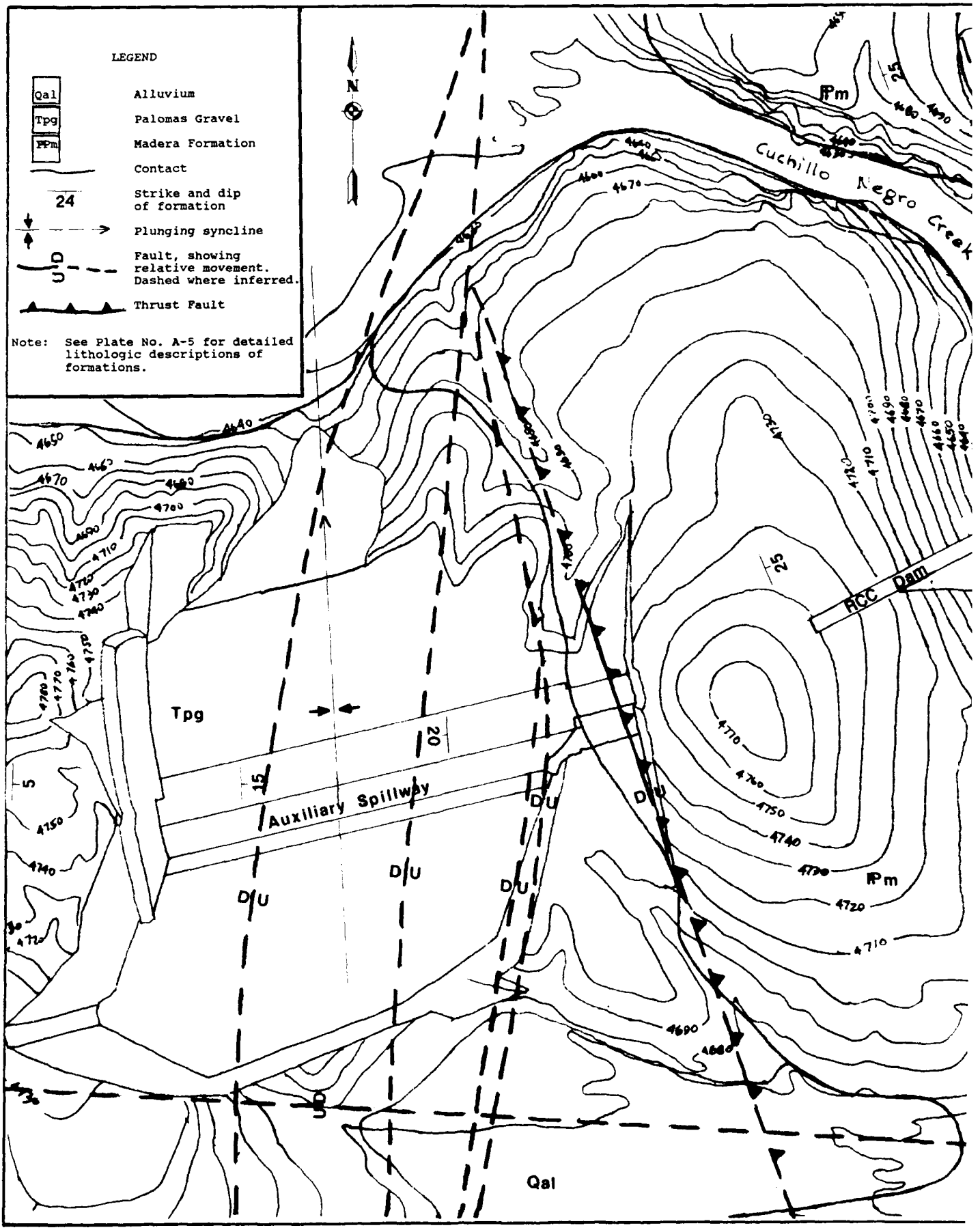
III2 North side of Cuchillo Negr Creek

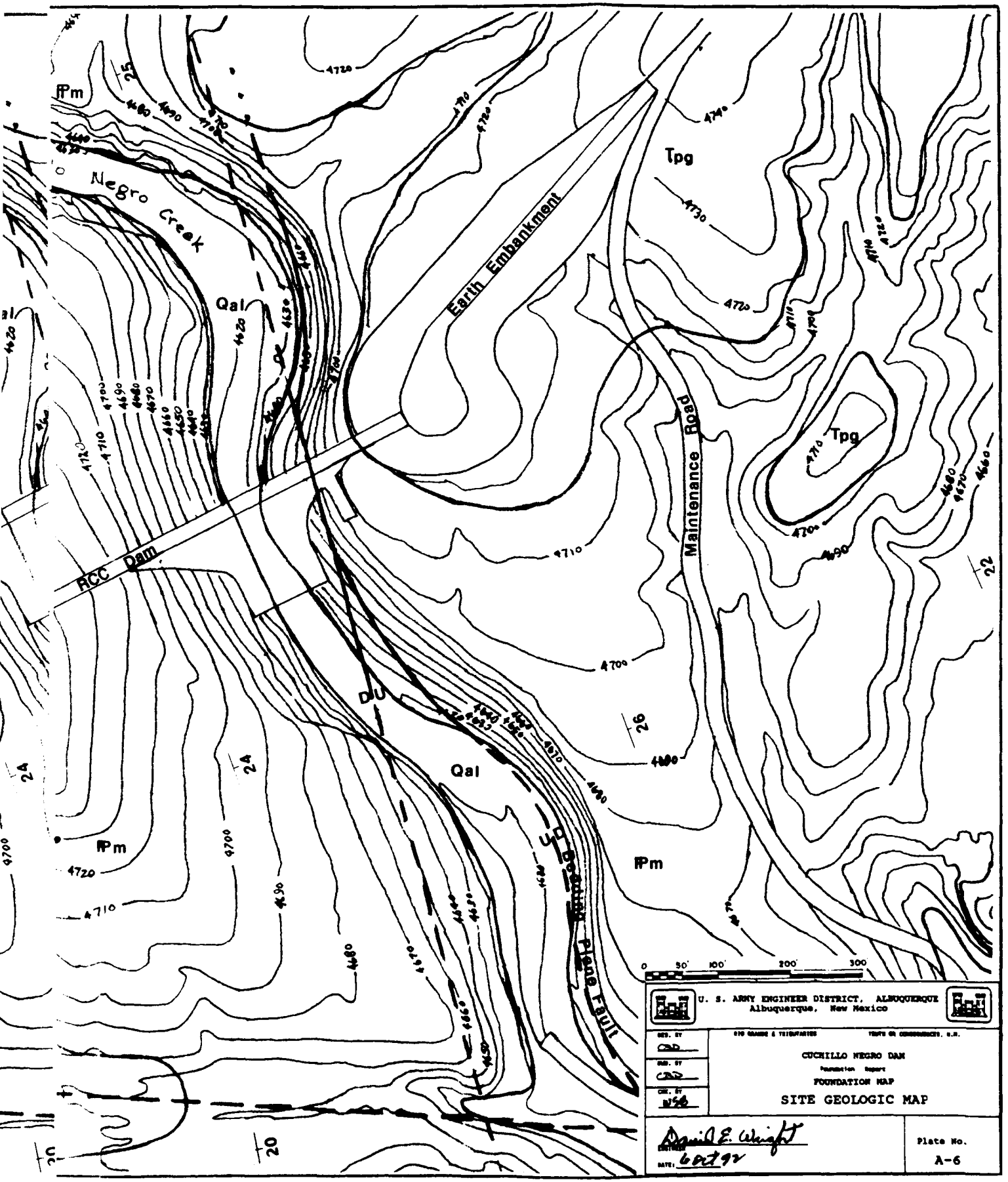
▯▯3 South side of Cuchillo Negr Creek

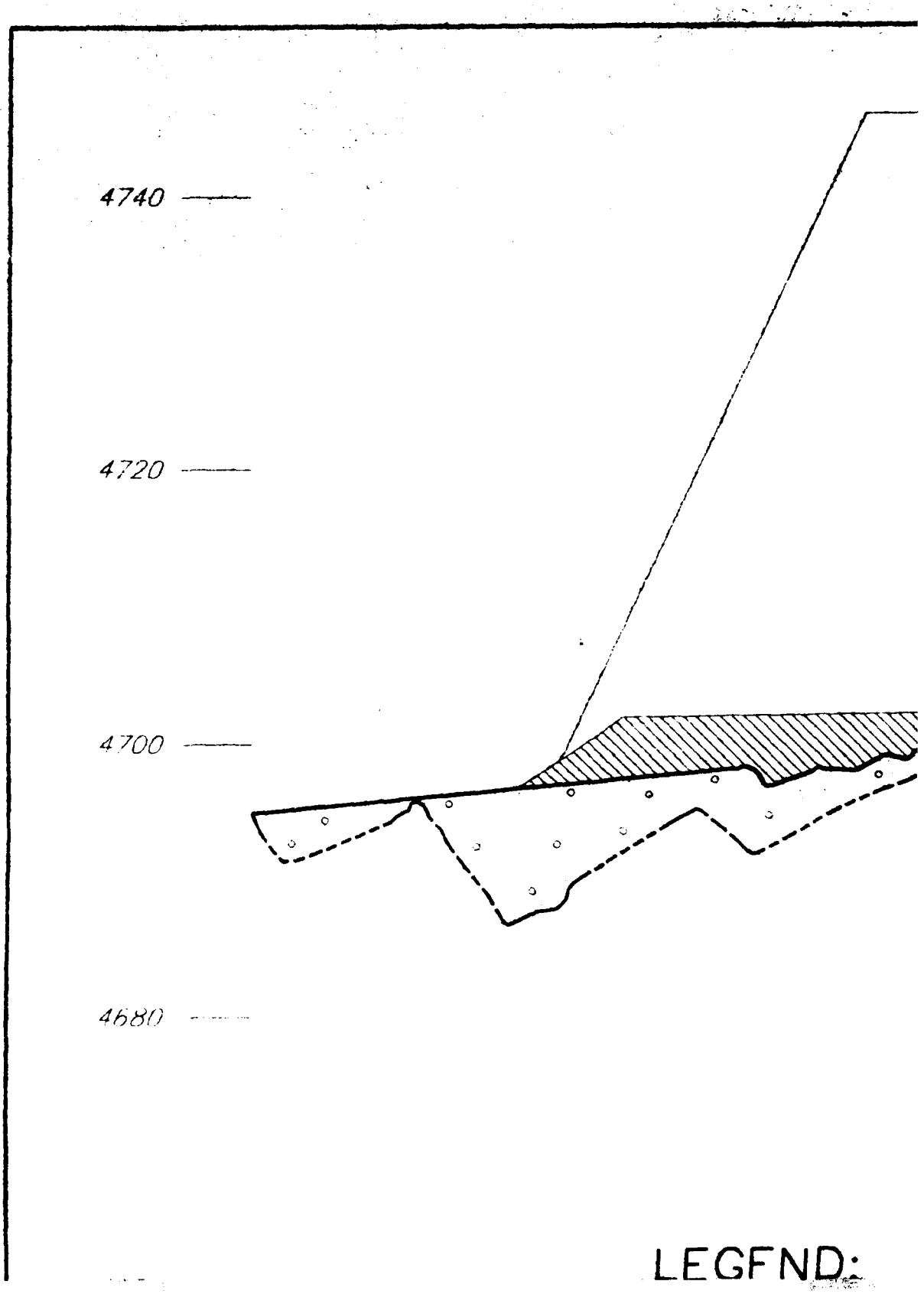
relatio am showing relationships of the Pliocene Cuchillo pediment surface and Pleistocene terraces along Cuchillo Negro Creek

Plate No. A-5

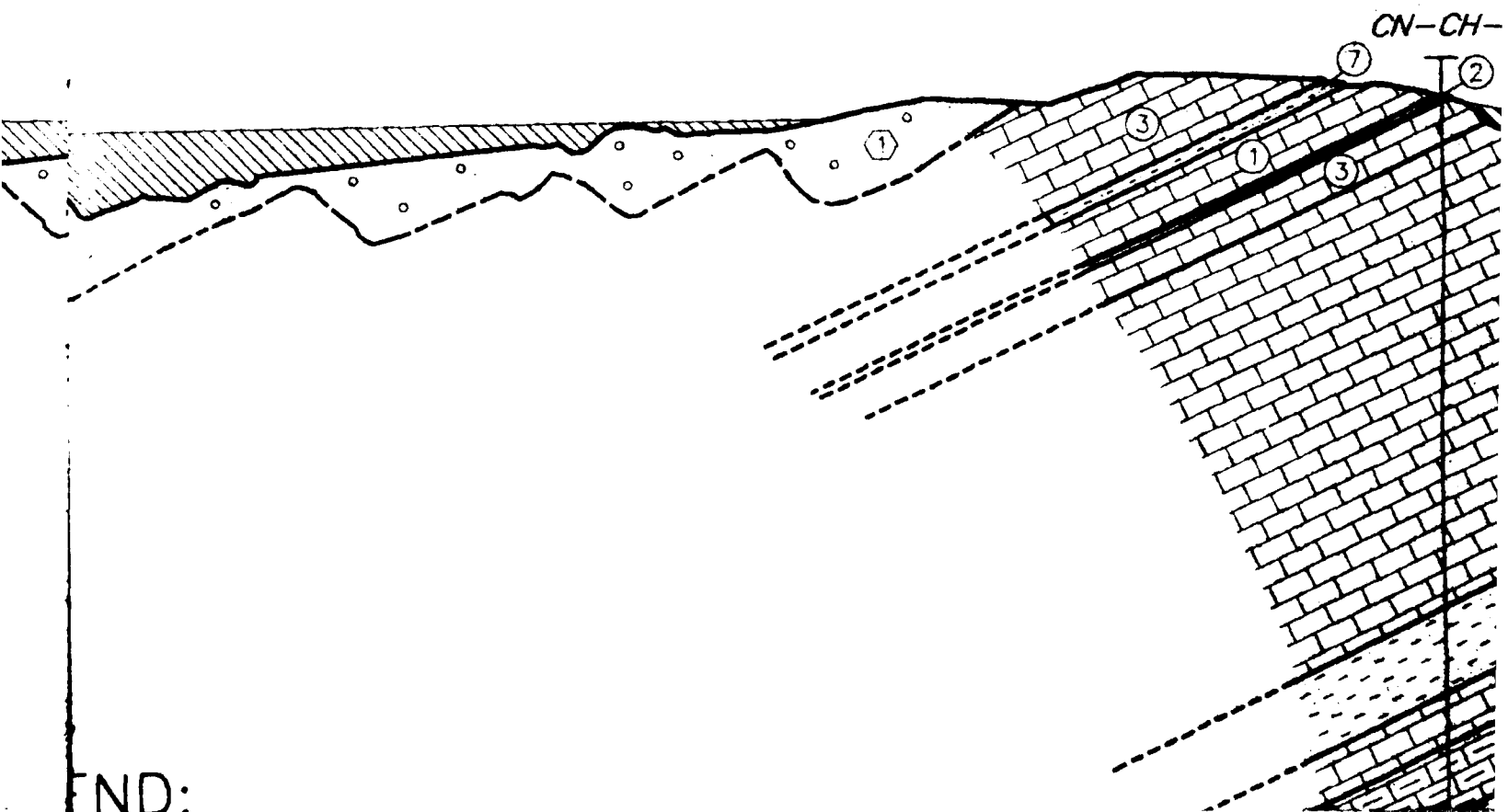
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LEGEND
Qal
Alluvium
Tpg
Palomas Gravel
Pm
Madera Formation
Contact
24
Strike and dip of formation
Plunging syncline
U D
Fault, showing relative movement. Dashed where inferred.
Thrust Fault
Note: See Plate No. A-5 for detailed lithologic descriptions of formations.
N
Cuchillo Negro Creek
RCC Dam
Auxiliary Spillway
Tpg
Pm
Qal
D U

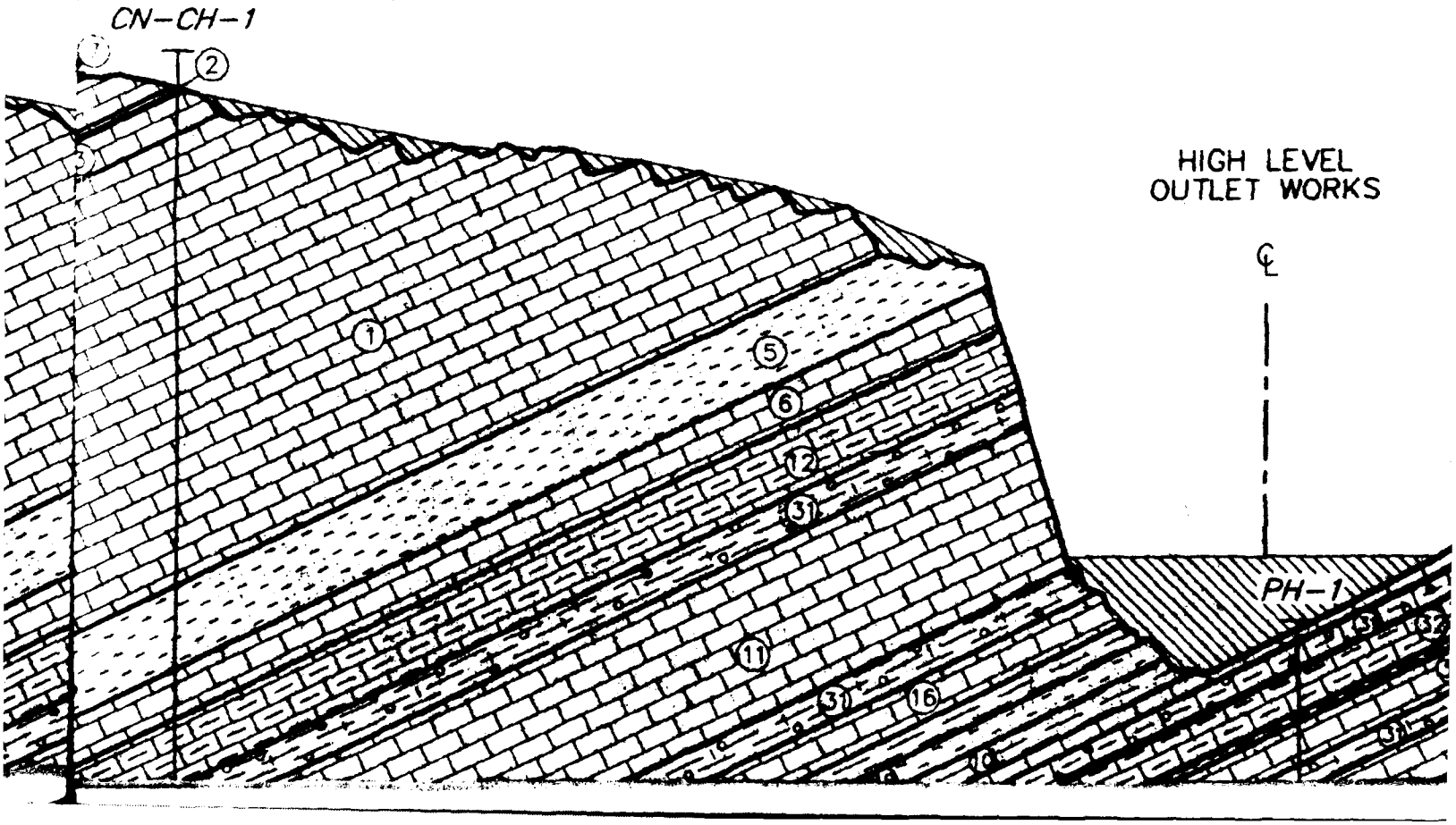

Negro Creek
Earth Embankment
Maintenance Road
RCC Dam
Tpg
Qal
Pm
DU
Jones Bosque Plane Fault
0 50' 100' 200' 300'
U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
Albuquerque, New Mexico
CUCHILLO NEGRO DAM
Foundation Report
FOUNDATION MAP
SITE GEOLOGIC MAP
David E. Wright
6 Oct 92
Plate No.
A-6



LEGFND:



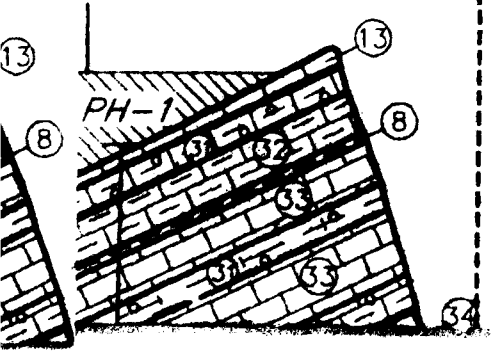
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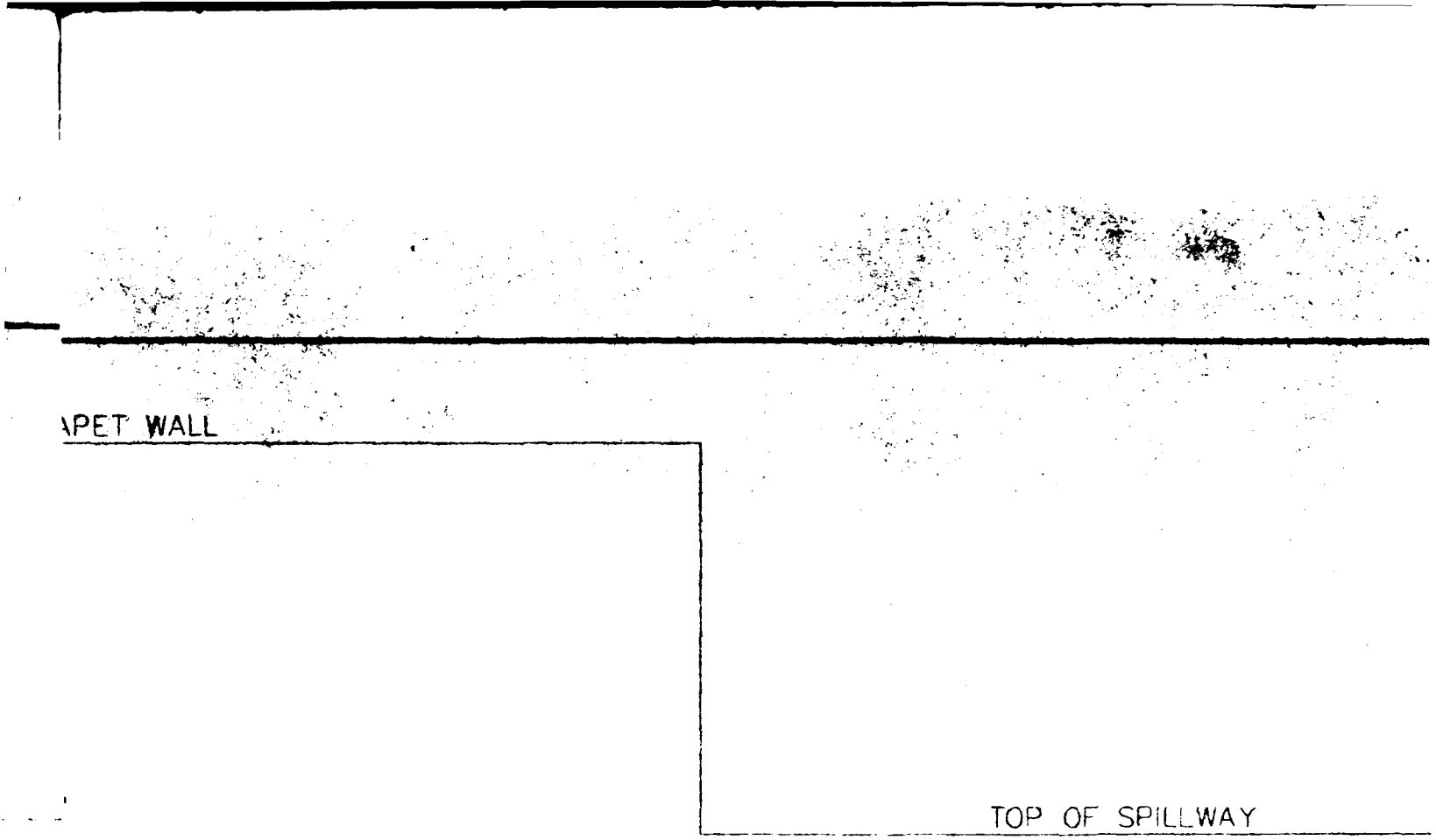

CN-CH-1
HIGH LEVEL
OUTLET WORKS
PH-1

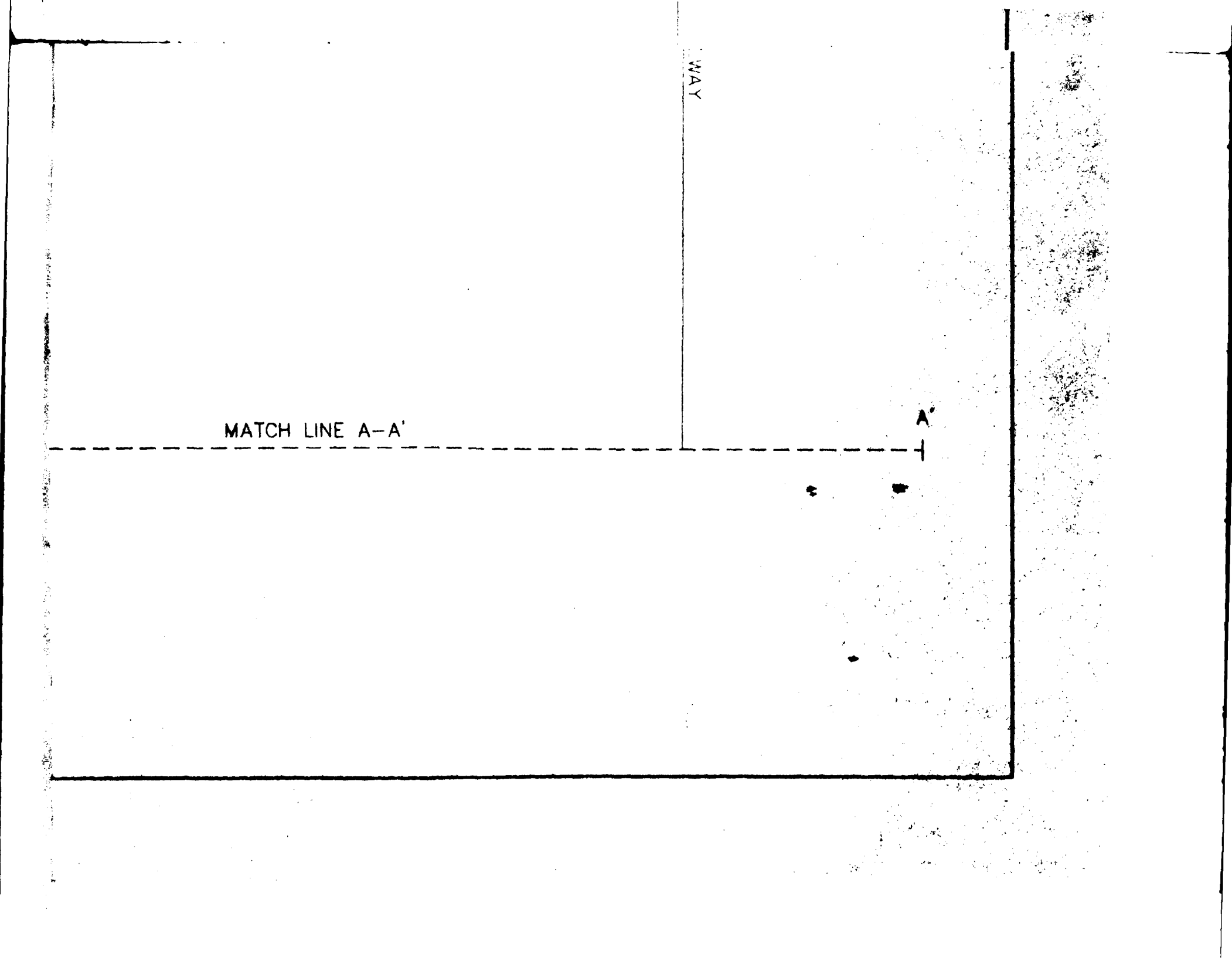
TOP OF PARAPET WALL

LEVEL
WORKS

CONTROL JOINT

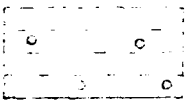


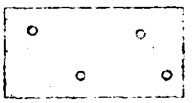

APET WALL
TOP OF SPILLWAY


WAY
MATCH LINE A–A'
A'

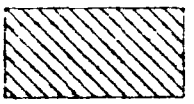
LEGEND:

4660

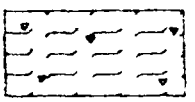

UNCONSOLIDATED AL
SILT, SAND, GRAVEL


CONGLOMERATE / C

4640

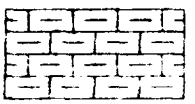

DENTAL CONCRETE

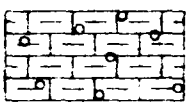

COLLAPSE BRECCIA
& INFILLING MATL.


SHEAR / BRECCIA

4620


LIMESTONE


ARGILLACEOUS LIMES

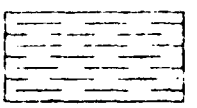

SHALE / LIMESTONE

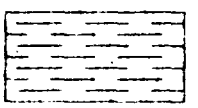
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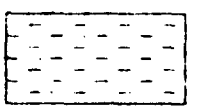

LIMESTONE & SHALE


LIMEY SHALE

4580

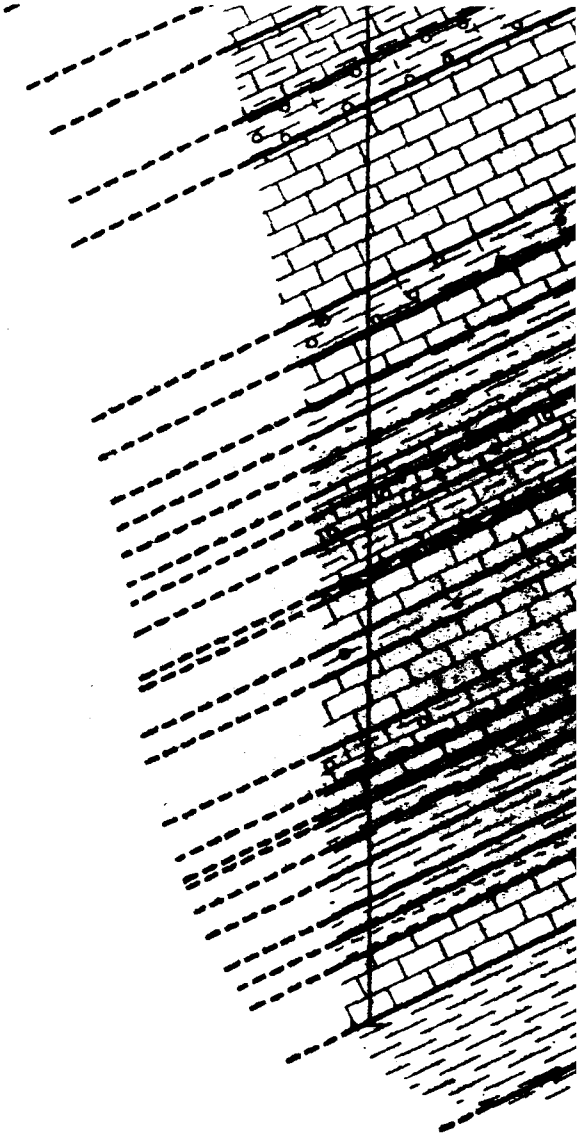

SHALE


SHALEY SILTSTONE


SILTSTONE

4560


CLAYSTONE



D ALLUVIUM
AVEL

/ CONGLOMERATE

RETE

CCIA
ATE.

CCIA

LIMESTONE

STONE

ONE

CONTACT BETWEEN MAPPED ROCK UNITS, DASHED WHERE ASSUMED

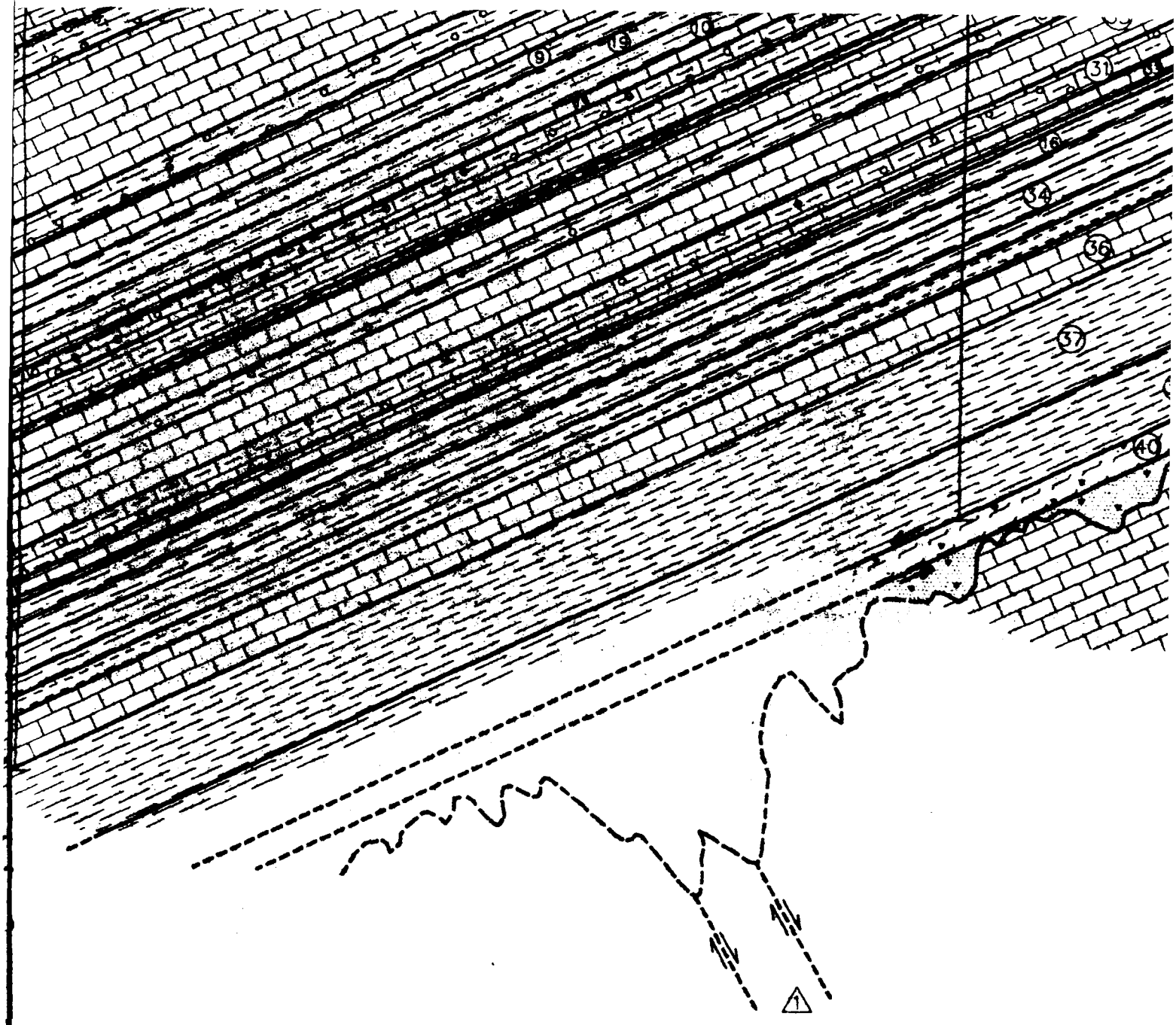
FAULTS SHOWING RELATIVE MOVEMENTS, DASHED WHERE ASSUMED

FINAL EXCAVATED SURFACE

(1) MAPPED UNIT–CORRESPONDS WITH NUMBER IN TABLE C–1

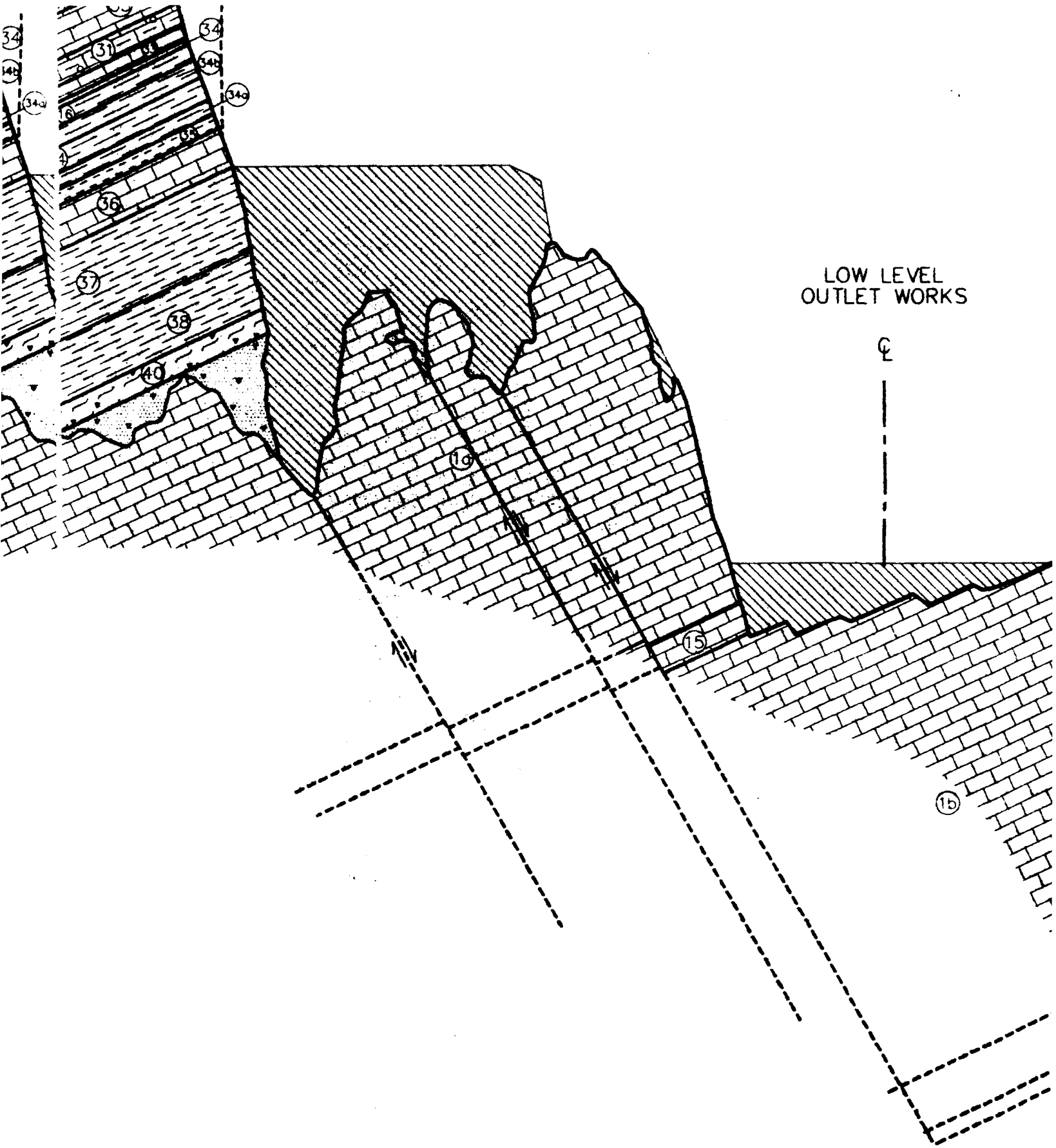
(5) MAPPED UNIT–CORRESPONDS WITH NUMBER IN TABLE C–2

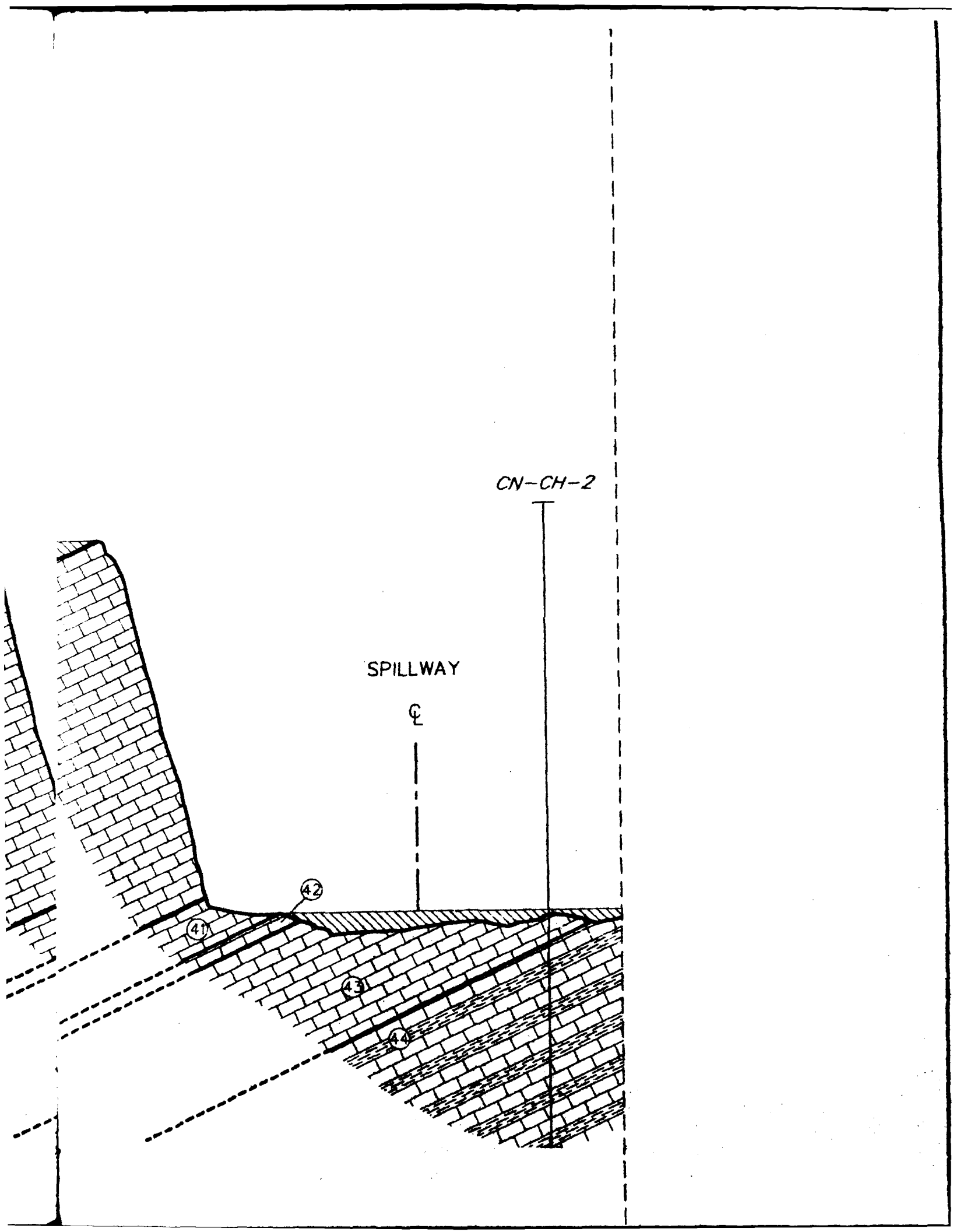
(1) NOTE:
FAULT LOCATIONS ESTIMATED BASED ON OBSERVED OUTCROPS DOWNSTREAM OF THE PROJECT

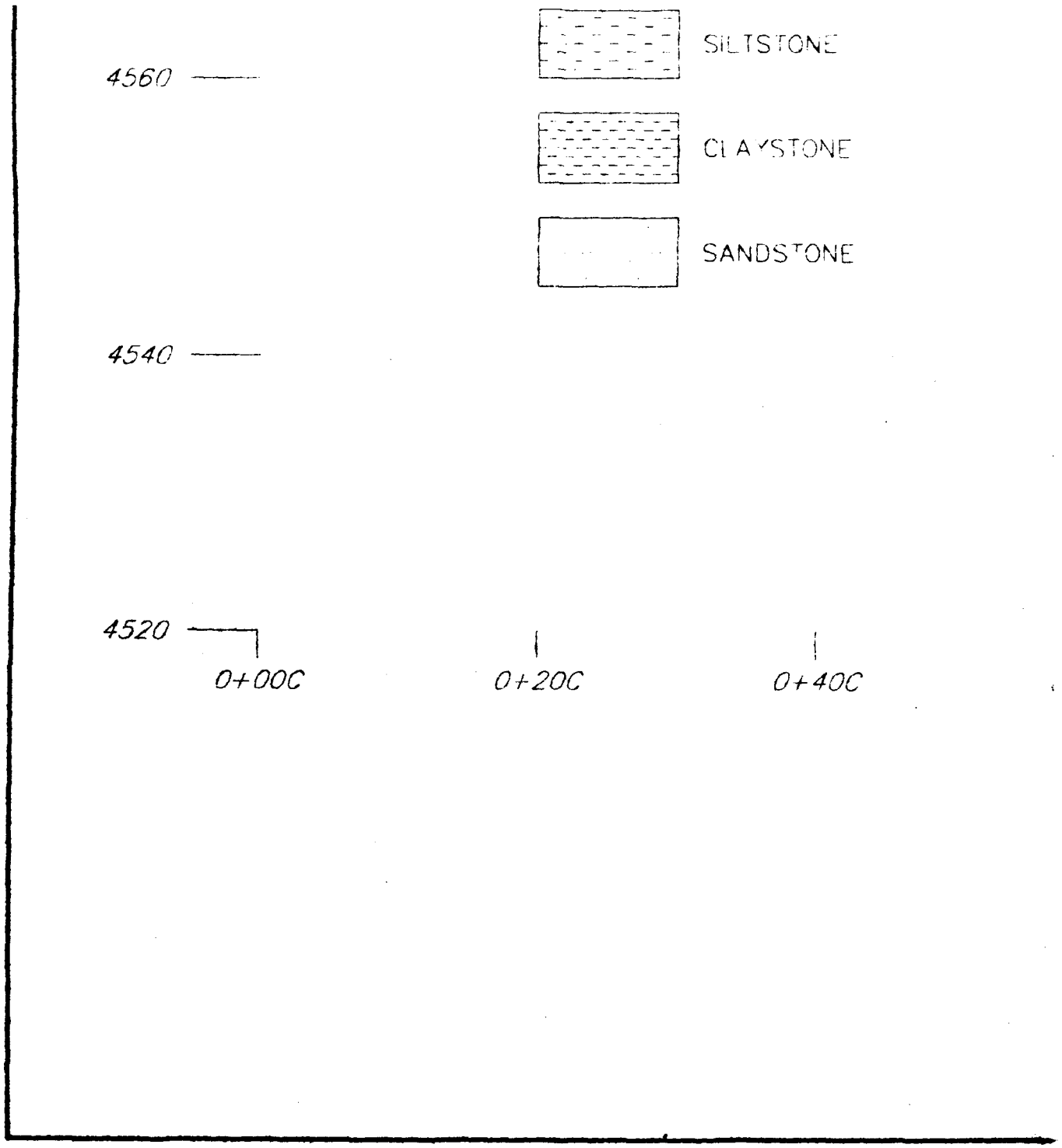


SCALE IN FEET:

10 5 0 10 20

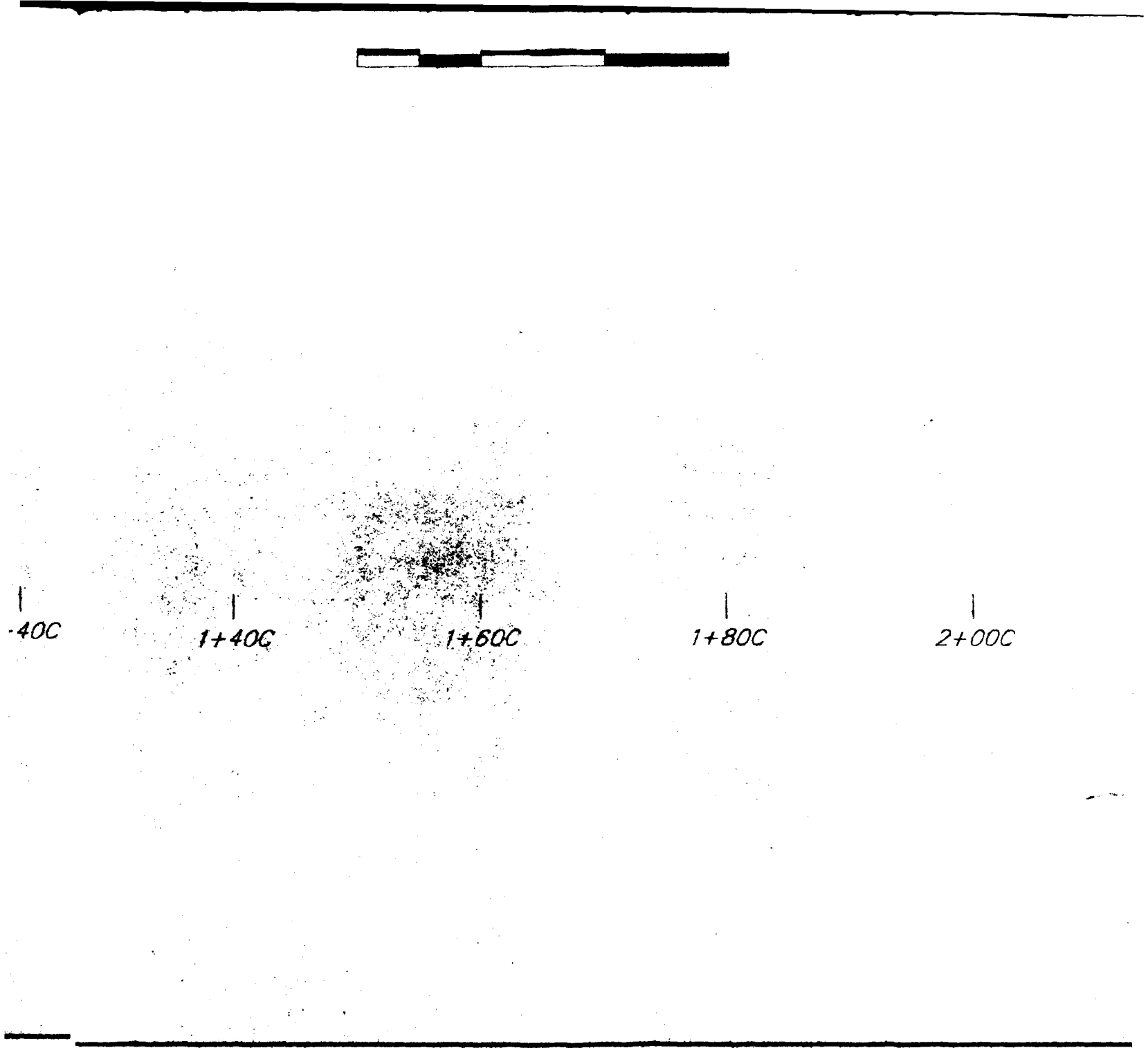

LOW LEVEL
OUTLET WORKS
℄
34
34b
34a
31
16
35
36
37
38
40
1a
15
1b

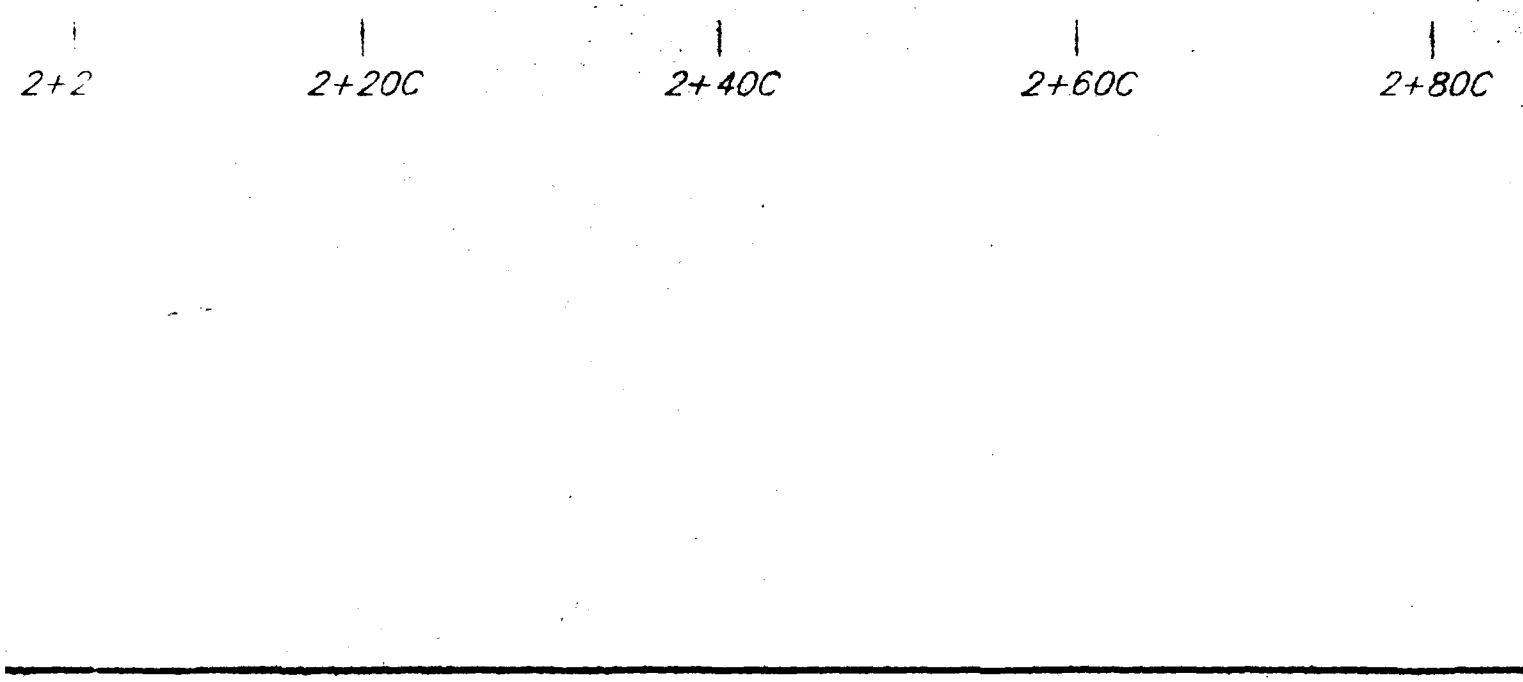

CN-CH-2
SPILLWAY
℄
42
41
43
44


SILTSTONE
CLAYSTONE
SANDSTONE
4560
4540
4520
0+00C
0+20C
0+40C

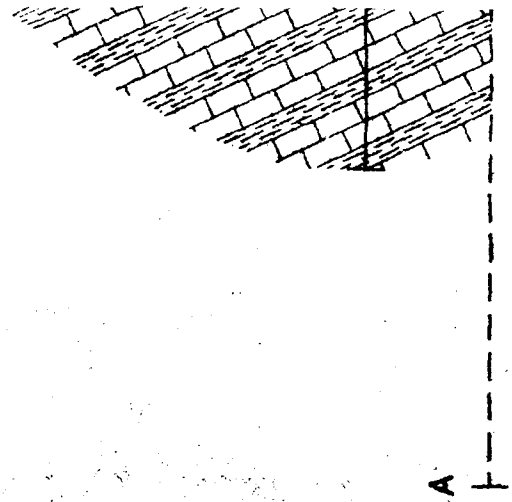
⚠ NOTE:
FAULT LOCATIONS ESTIMATED
BASED ON OBSERVED OUTCROPS
DOWNSTREAM OF THE PROJECT

0+60C | 0+80C | 1+00C | 1+20C


-40C
1+40C
1+60C
1+80C
2+00C



DESIGNE
C.E
DRAWN
S.E
CHECKE
W.S
ENGI
DATE:



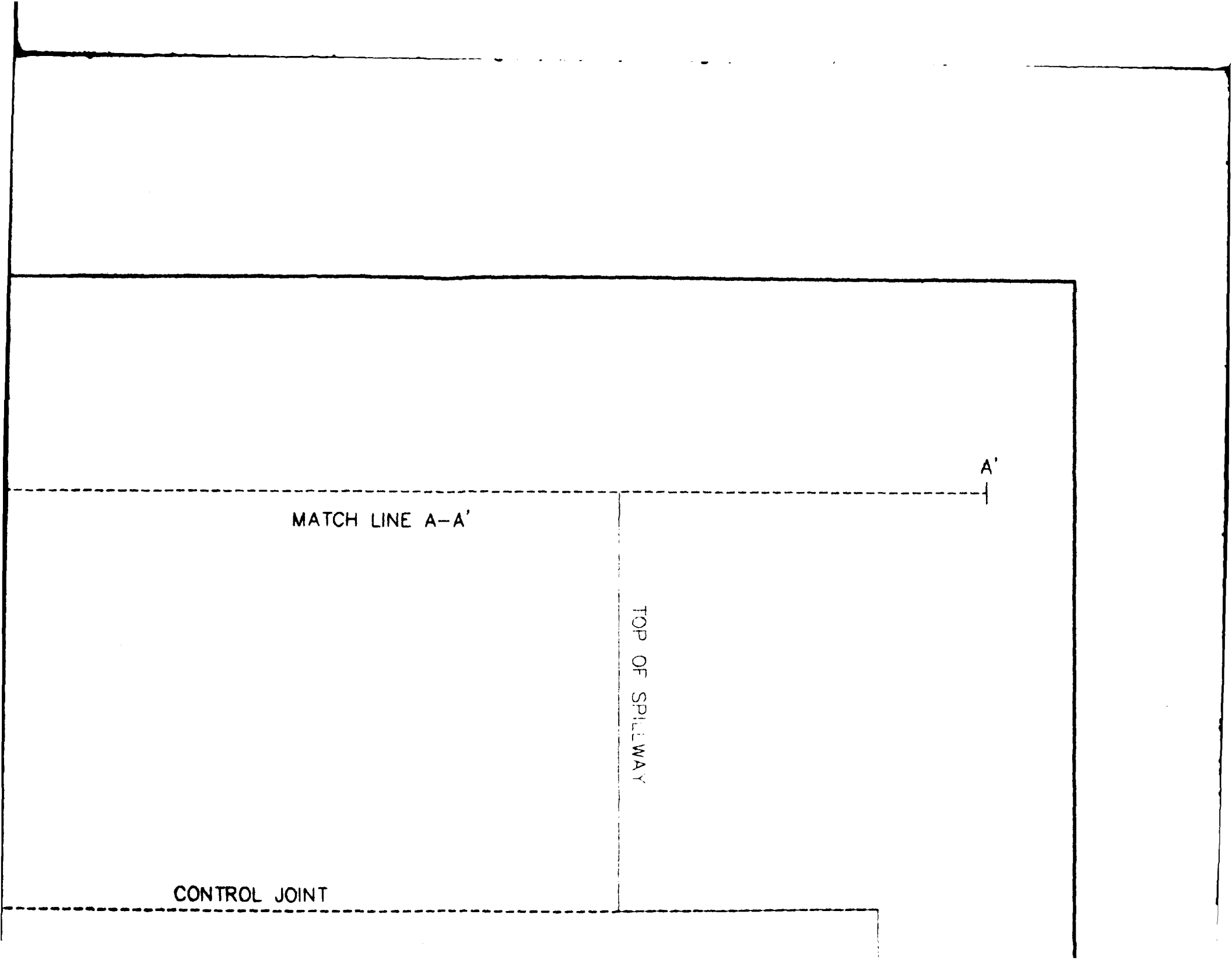
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CORPS OF ENGINEERS
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DESIGNED BY: C.B.D.	RIO GRANDE & TRIBUTARIES	TRUTH OR CONSEQUENCES, N.M.
DRAWN BY: S.D.M.	CUCHILLO NEGRO DAM FOUNDATION REPORT	
CHECKED BY: W.S.B.	**GEOLOGIC CROSS SECTION ALONG RCC DAM AXIS LOOKING S27°E**	

David E. Wright
ENGINEER
DATE: 5 oct 92

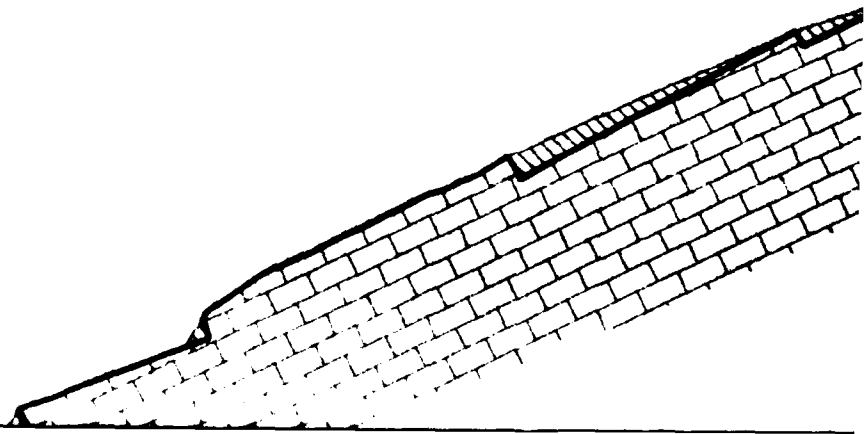
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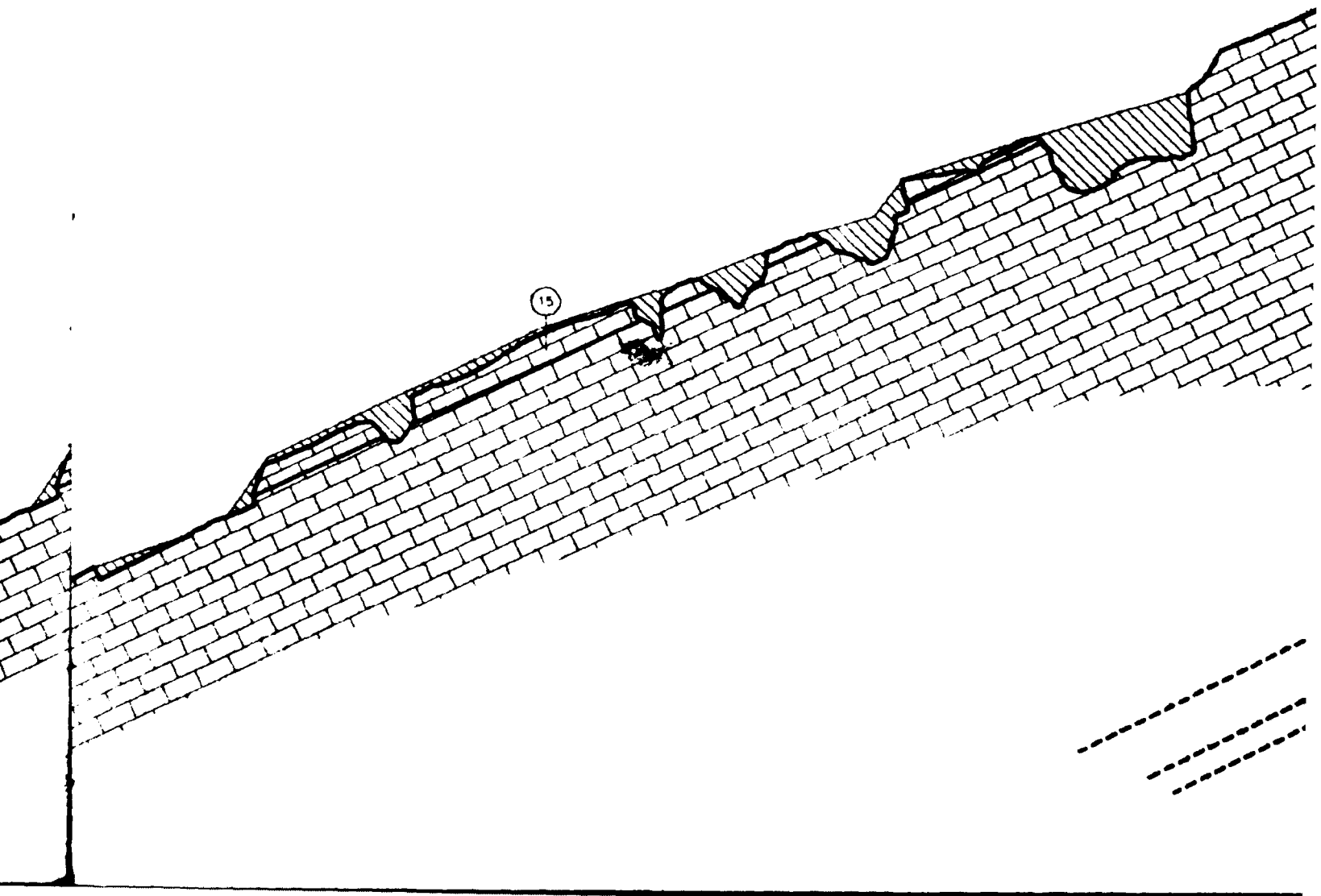
PLATE
A-7

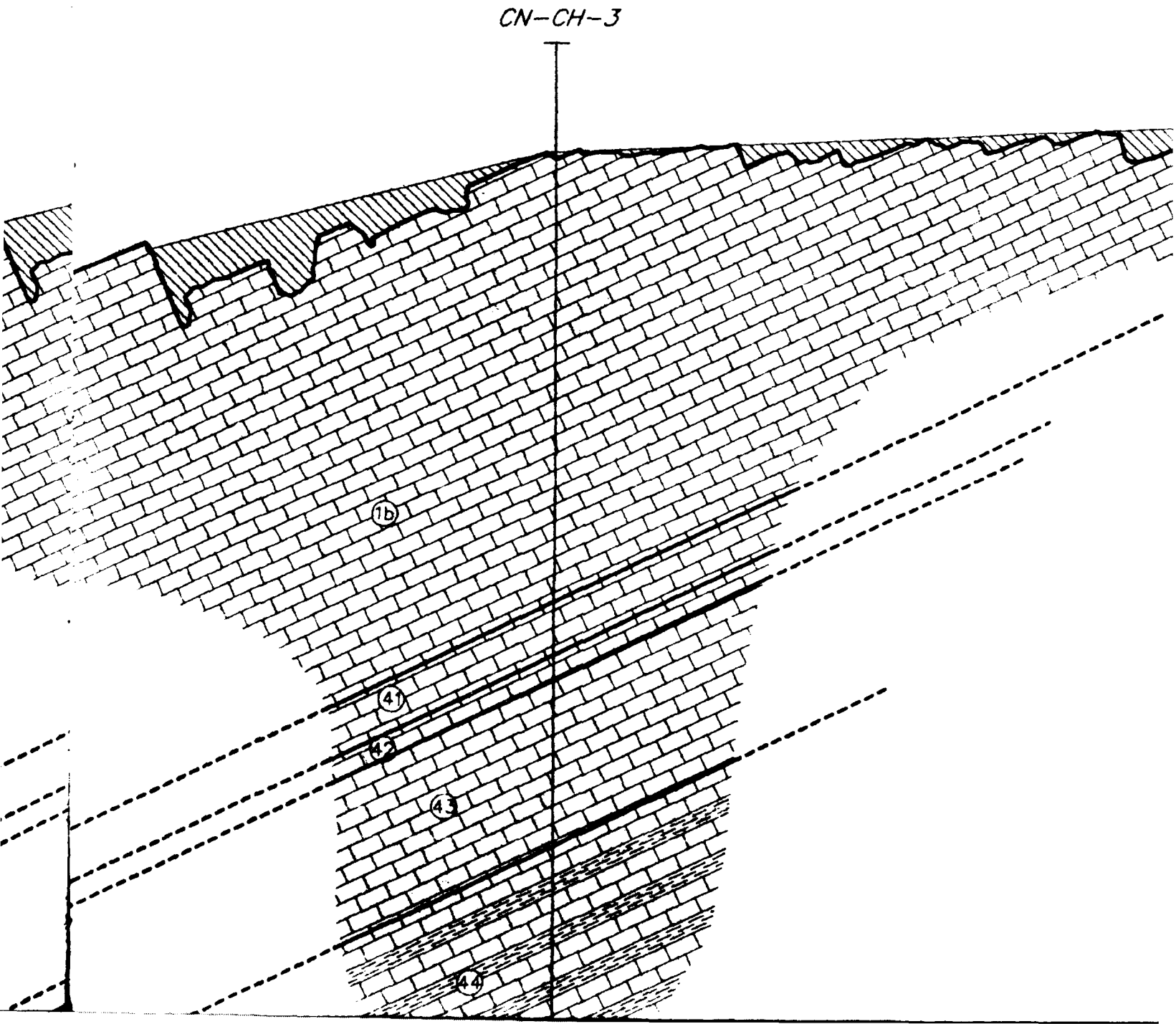

A'
MATCH LINE A–A'
TOP OF SPILLWAY
CONTROL JOINT

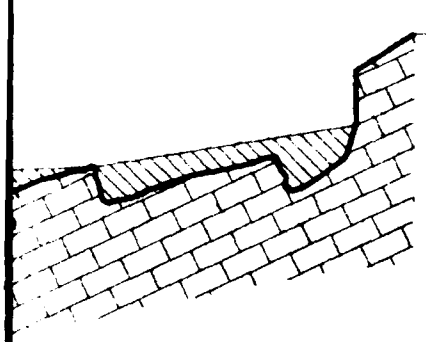
TOP OF PARAPET WALL

CONTROL JOINT




15


CN-CH-3
1b
41
42
43
44

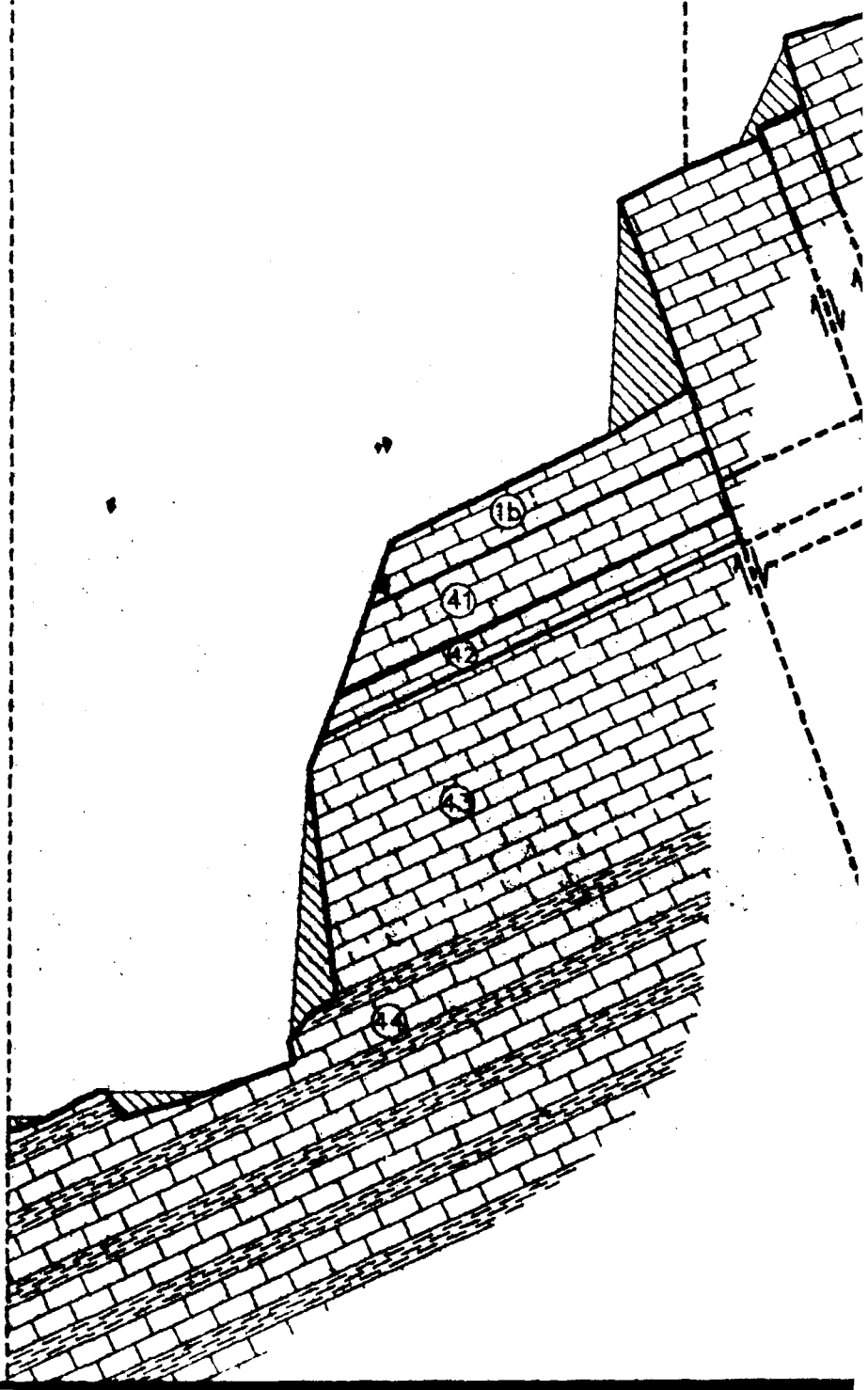


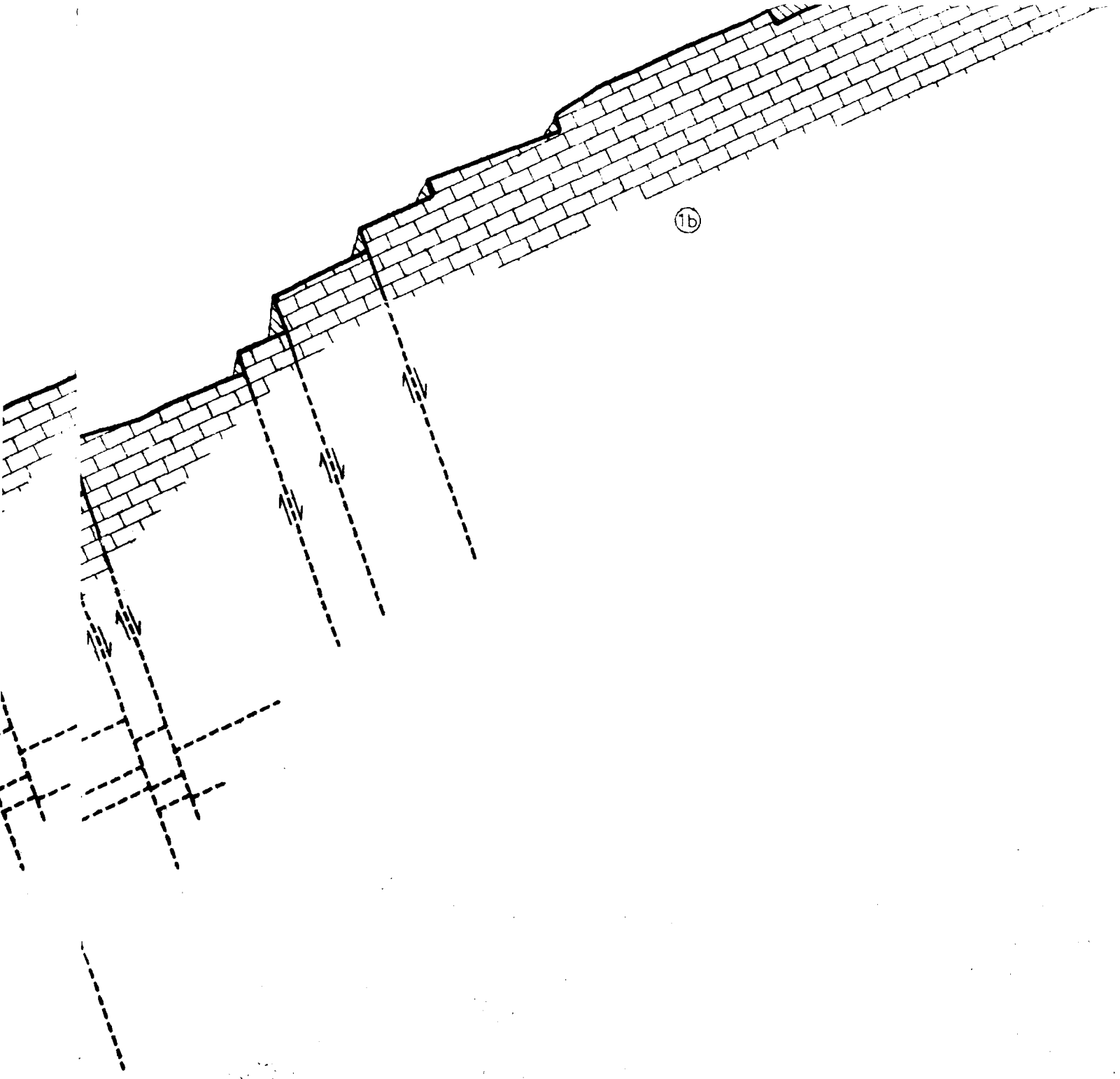
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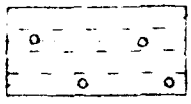
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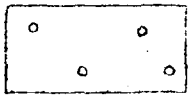
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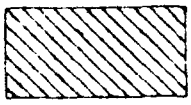

1b
41
42
43
44

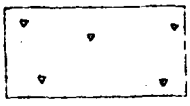

1b

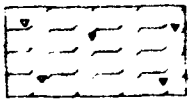
LEGEND:


UNCONSOLIDATED ALLUVIUM
SILT, SAND, GRAVEL


FANGLOMERATE / CONGLOMERATE

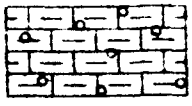

DENTAL CONCRETE


COLLAPSE BRECCIA
& INFILLING MATL.

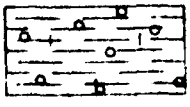

SHEAR / BRECCIA

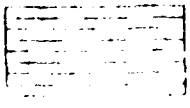

LIMESTONE


ARGILLACEOUS LIMESTONE

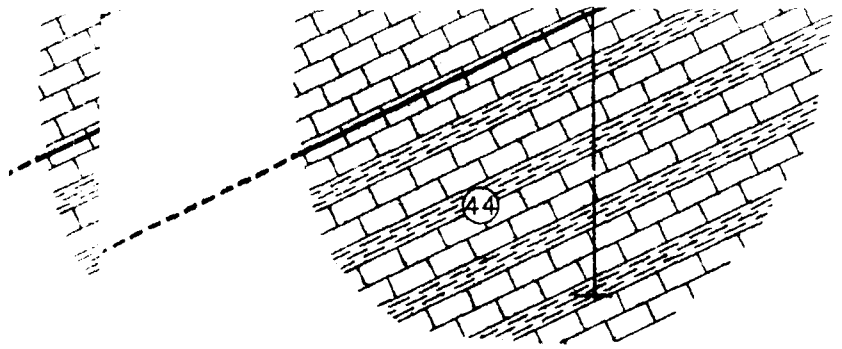

SHALE / LIMESTONE


LIMESTONE & SHALE


LIMEY SHALE


SHALE





ACT B_ CONTACT BETWEEN MAPPED ROCK
S, DAS_ UNITS, DASHED WHERE ASSUMED

TS SH FAULTS SHOWING RELATIVE MOVE-
S, DA MENTS, DASHED WHERE ASSUMED

EXCA FINAL EXCAVATED SURFACE

ED UN MAPPED UNIT--CORRESPONDS
NUMB WITH NUMBER IN TABLE C-1

MAPPED UNIT--CORRESPONDS

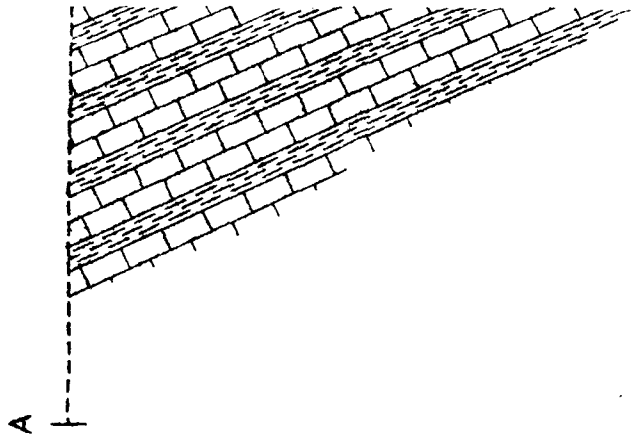
4660

4640

4620

4600

4580



3+40C

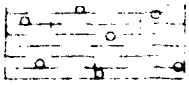
3+60C

3+80C

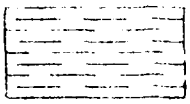
SCALE IN FEET:



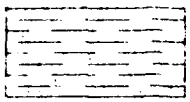
3+80C
4+00C
4+20C
4+40C
4+60C



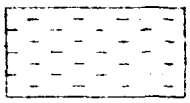
LIMEY SHALE



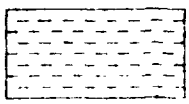
SHALE



SHALEY SILTSTONE



SILTSTONE



CLAYSTONE



SANDSTONE



FAULT
MENT

FINAL

① MAPP
WITH

⑤ MAPP
WITH

+60C

4+80C

5+00C

5+20C

5+40C

FAULTS SHOWING RELATIVE MOVEMENTS, DASHED WHERE ASSUMED

- FINAL EXCAVATED SURFACE

MAPPED UNIT–CORRESPONDS WITH NUMBER IN TABLE C–1

MAPPED UNIT–CORRESPONDS WITH NUMBER IN TABLE C–2

6+20C

OC

5+60C

5+80C

6+00C

DESIGNED BY: C.B.D.	RIO GR
DRAWN BY: S.D.M.	
CHECKED BY: W.S.B.	

David E. W

ENGINEER

DATE: 5 Sep 9

4560

6+40C

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

BY:
D.
Y:
M.
BY:
B.

RIO GRANDE & TRIBUTARIES — TRUTH OR CONSEQUENCES, N.M.

CUCHILLO NEGRO DAM

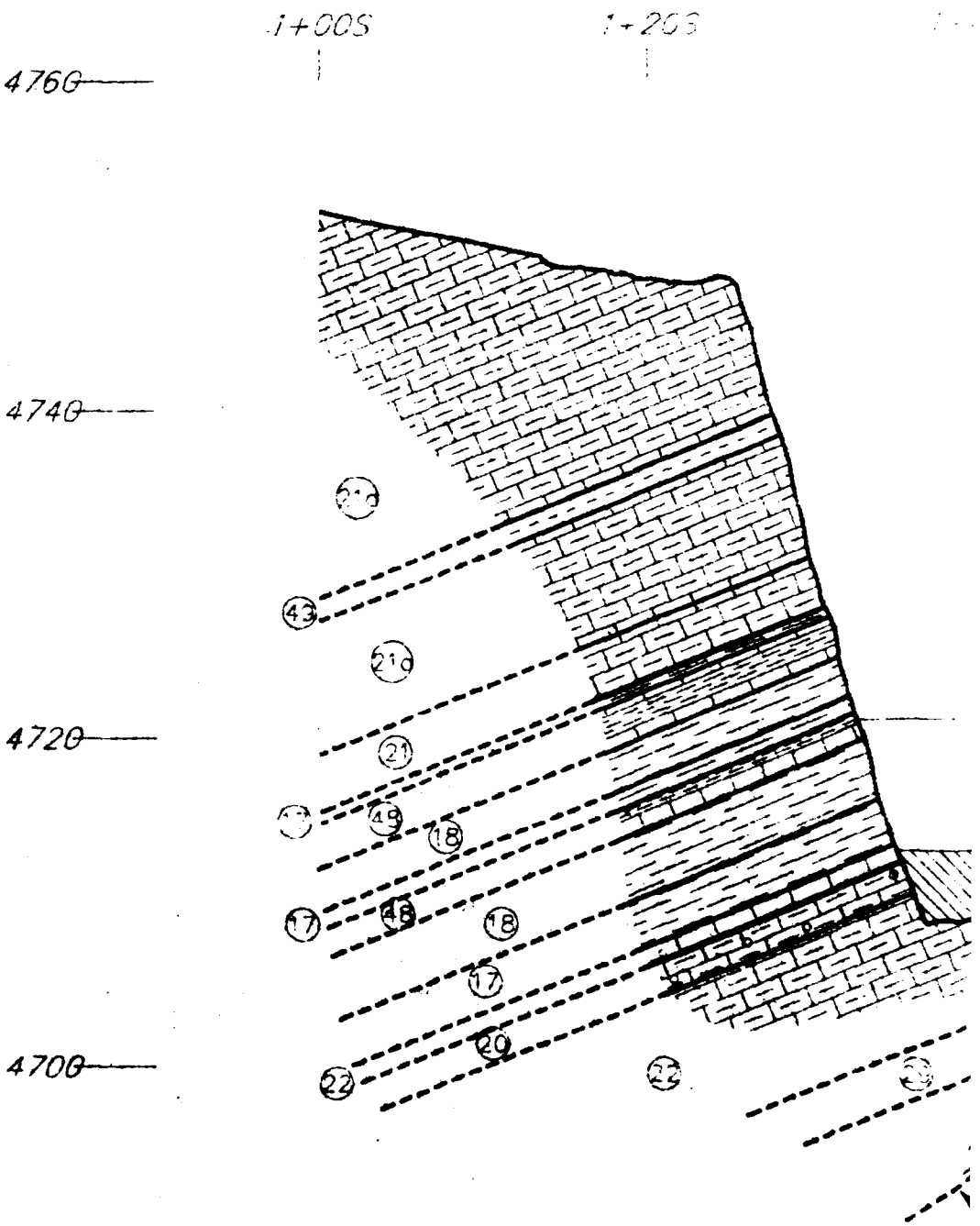
FOUNDATION REPORT

GEOLOGIC CROSS SECTION ALONG RCC DAM AXIS LOOKING S27°E

Harold E. Wright
EER
5 Sep 92

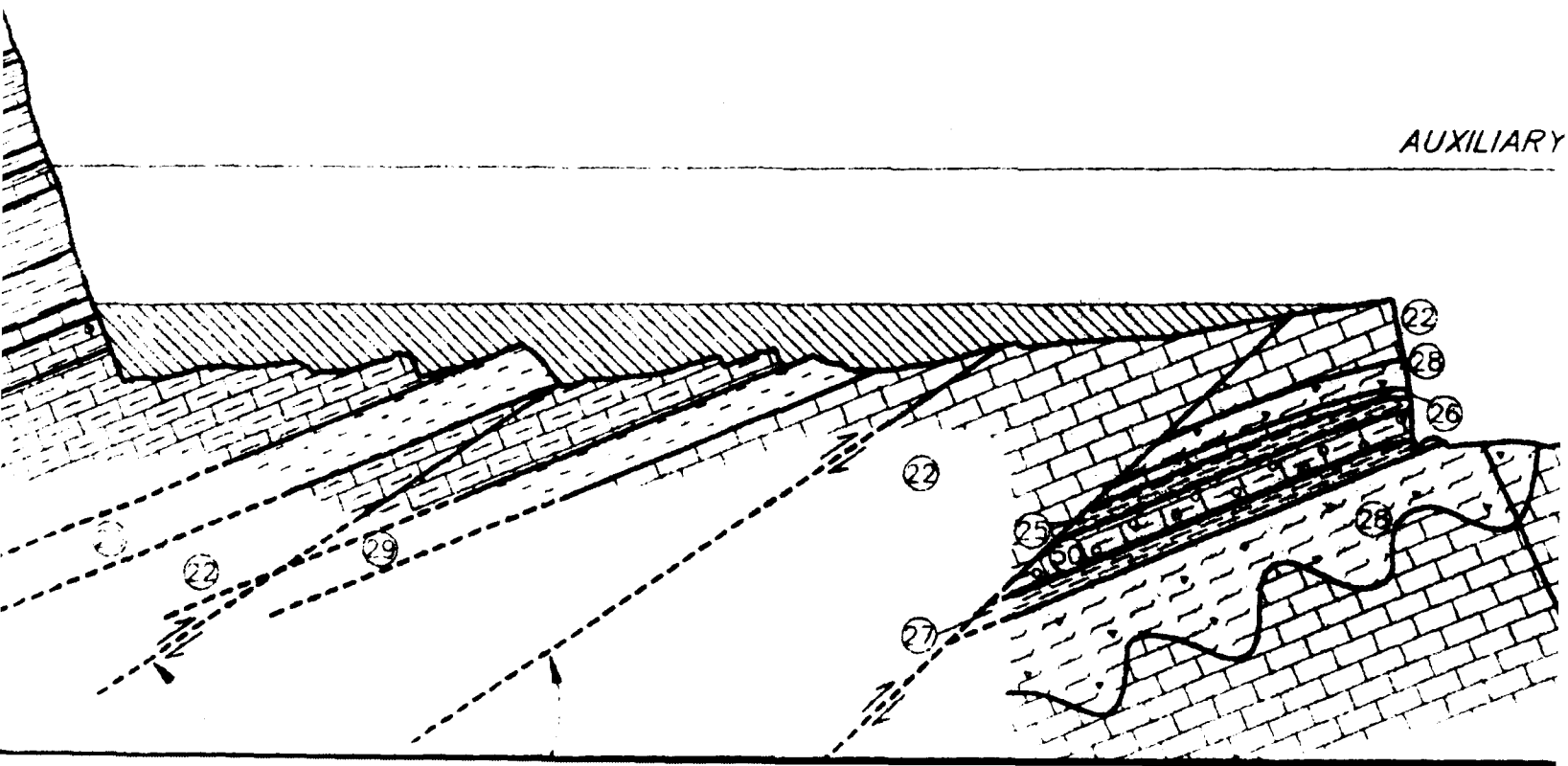
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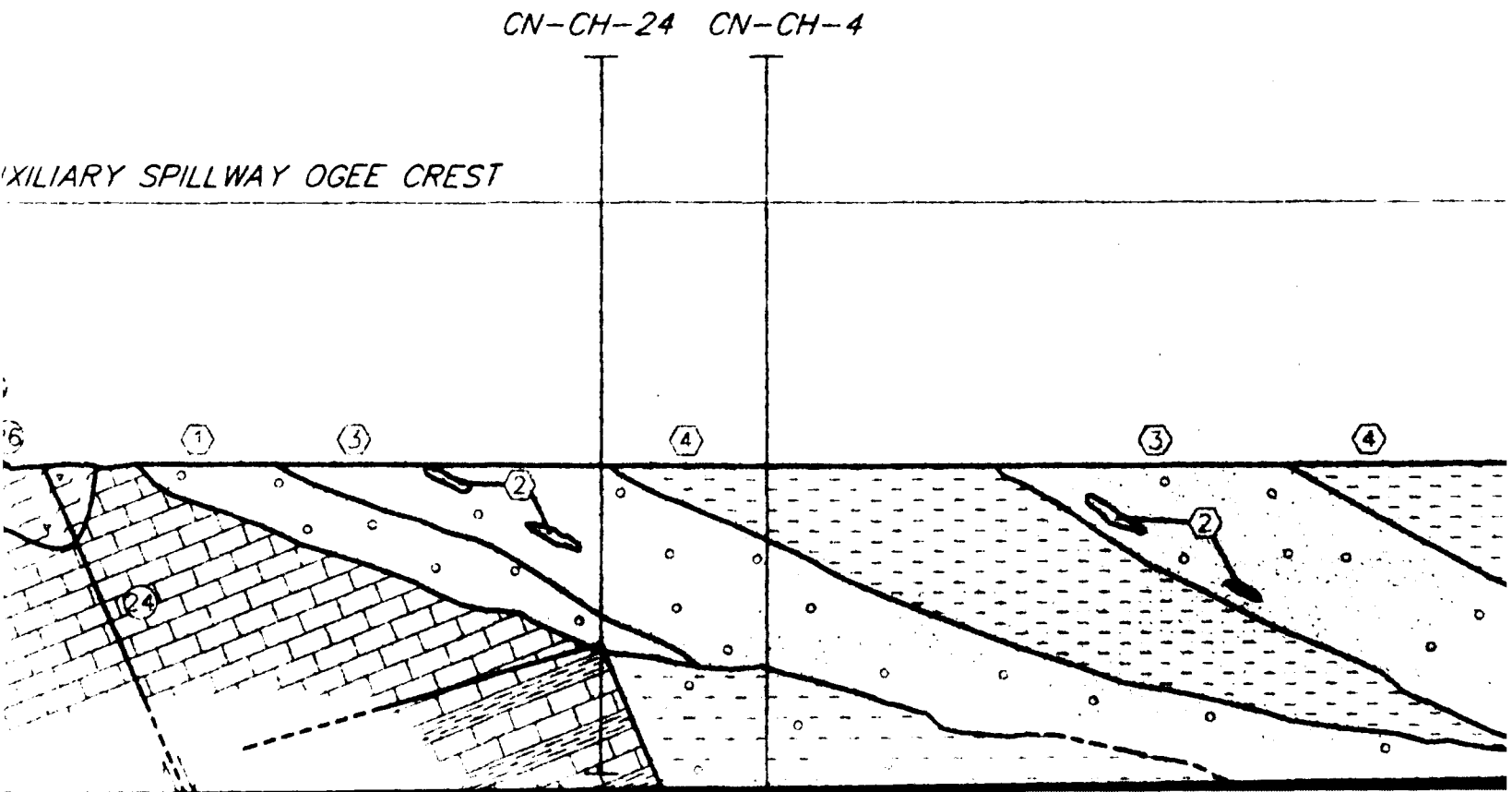

1+00S
1+20S
4760
4740
4720
4700
21a
43
21a
21
19
18
17
18
17
20
22
23

1+40S 1+60S 1+80S 2+00S 2+

AUXILIARY

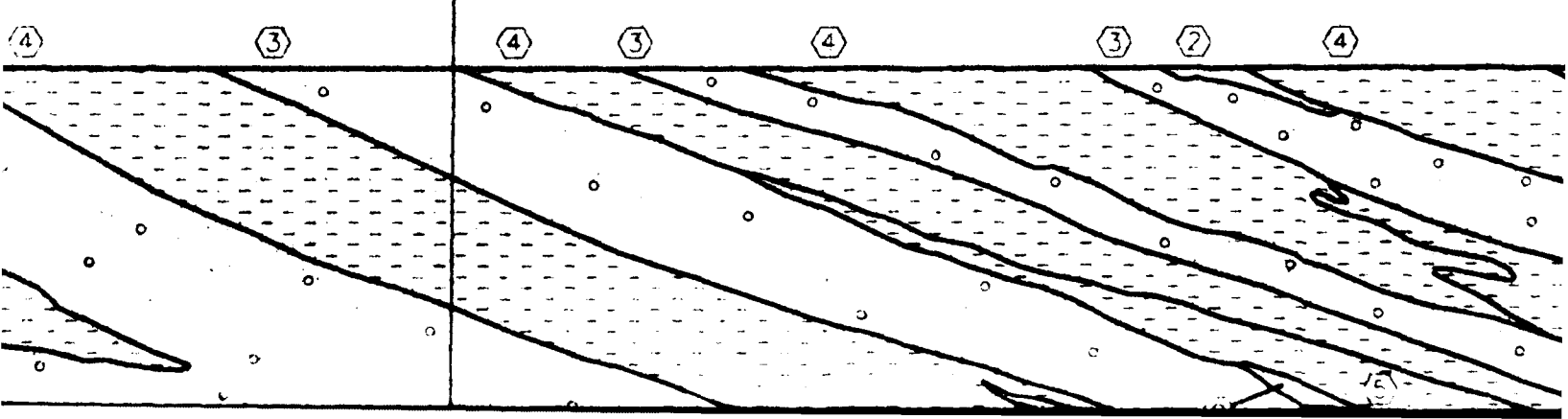


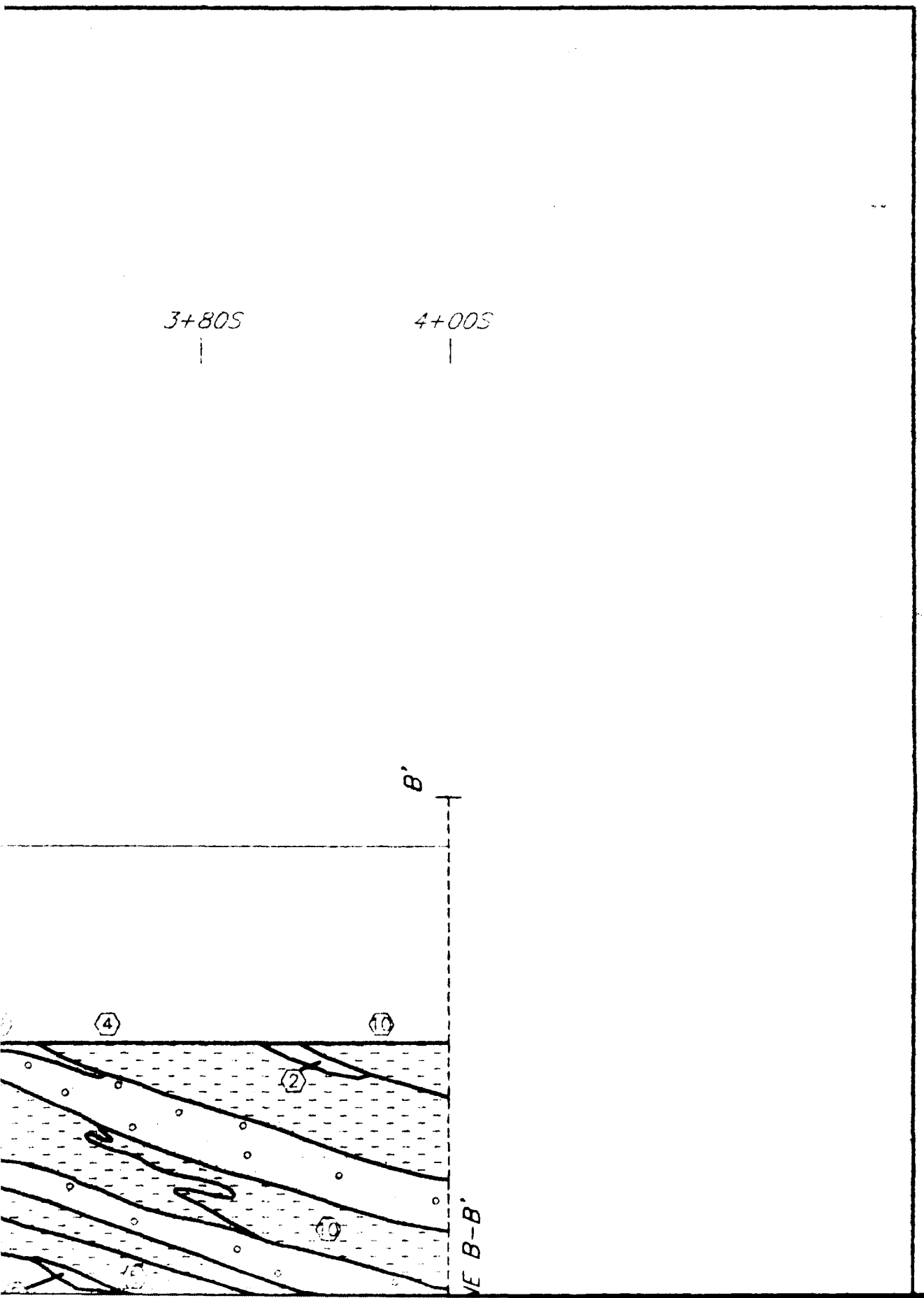
2+20S 2+40S 2+60S 2+80S 3+00S



3+00S 3+20S 3+40S 3+60S 3+80S

CN-CH-41




3+80S
4+00S
B'
4
10
2
10
E B-B'

4700

4680

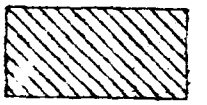
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4640

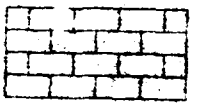
4620

4600

LEG



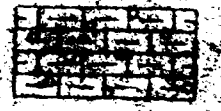
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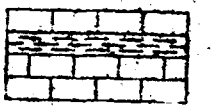
LIME



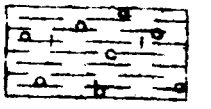
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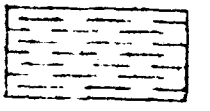
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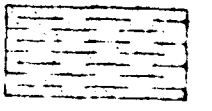
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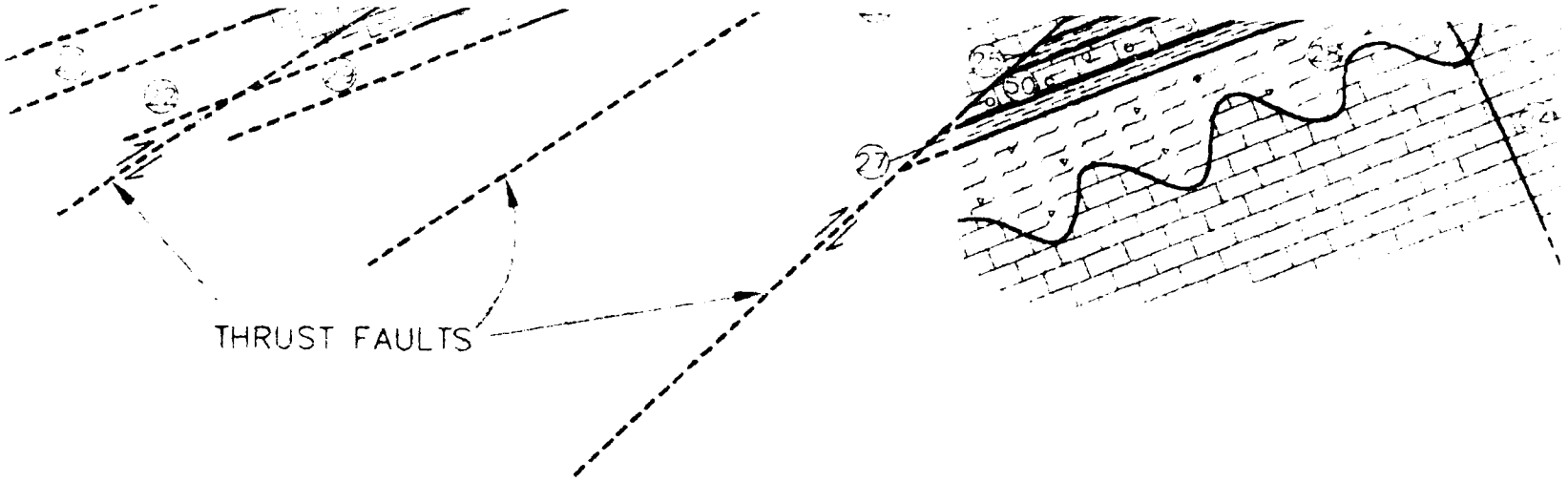
LIME



SHAL



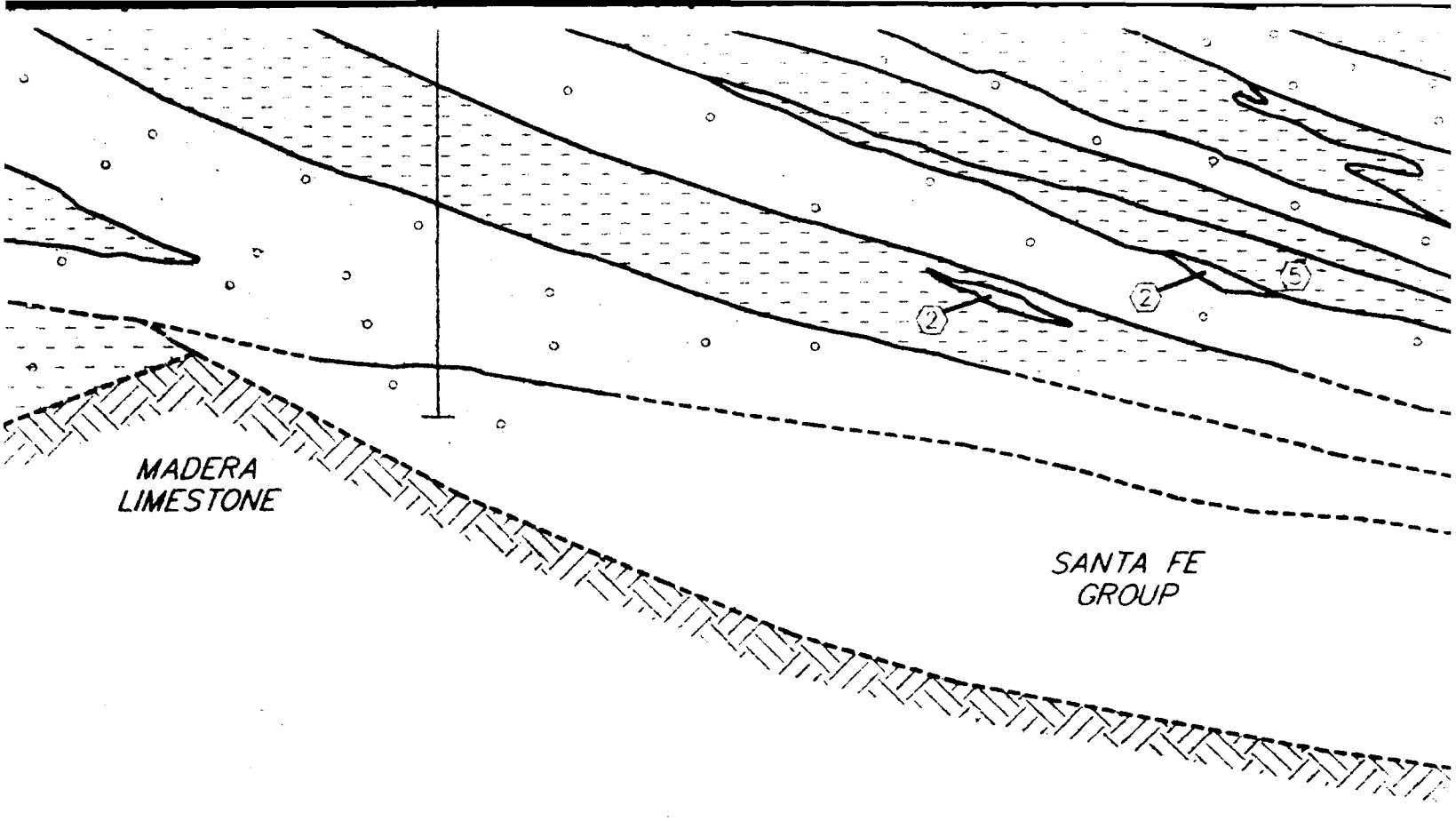
SHAL

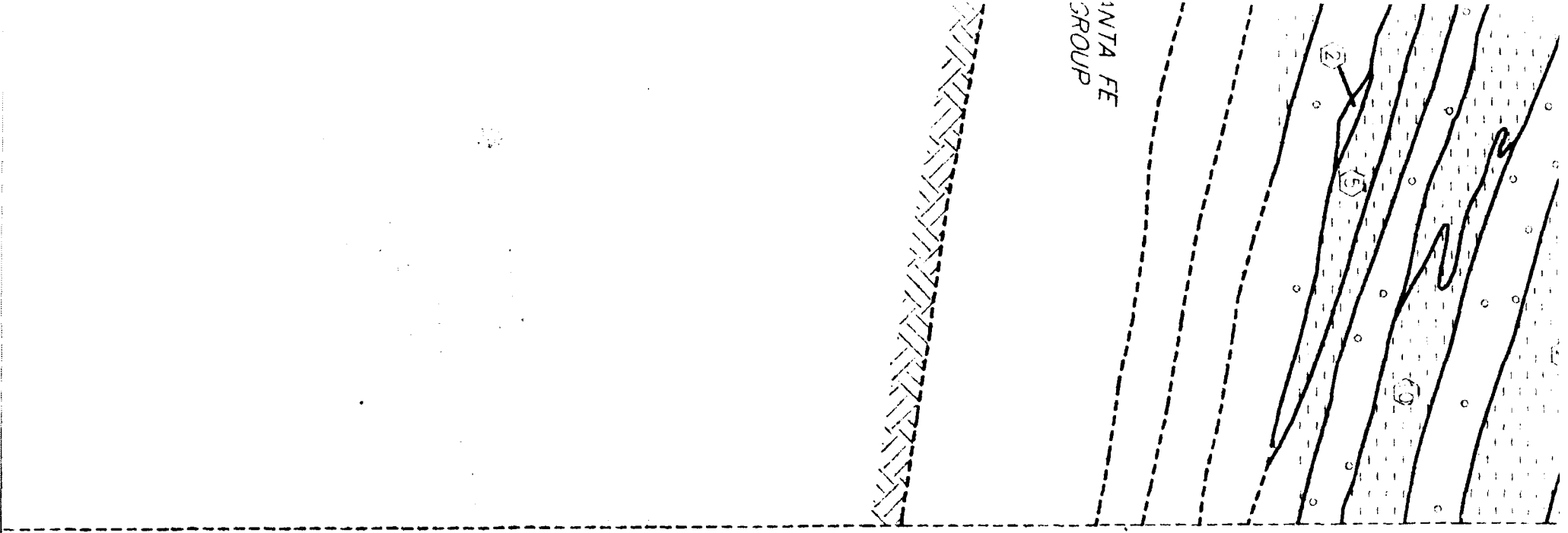


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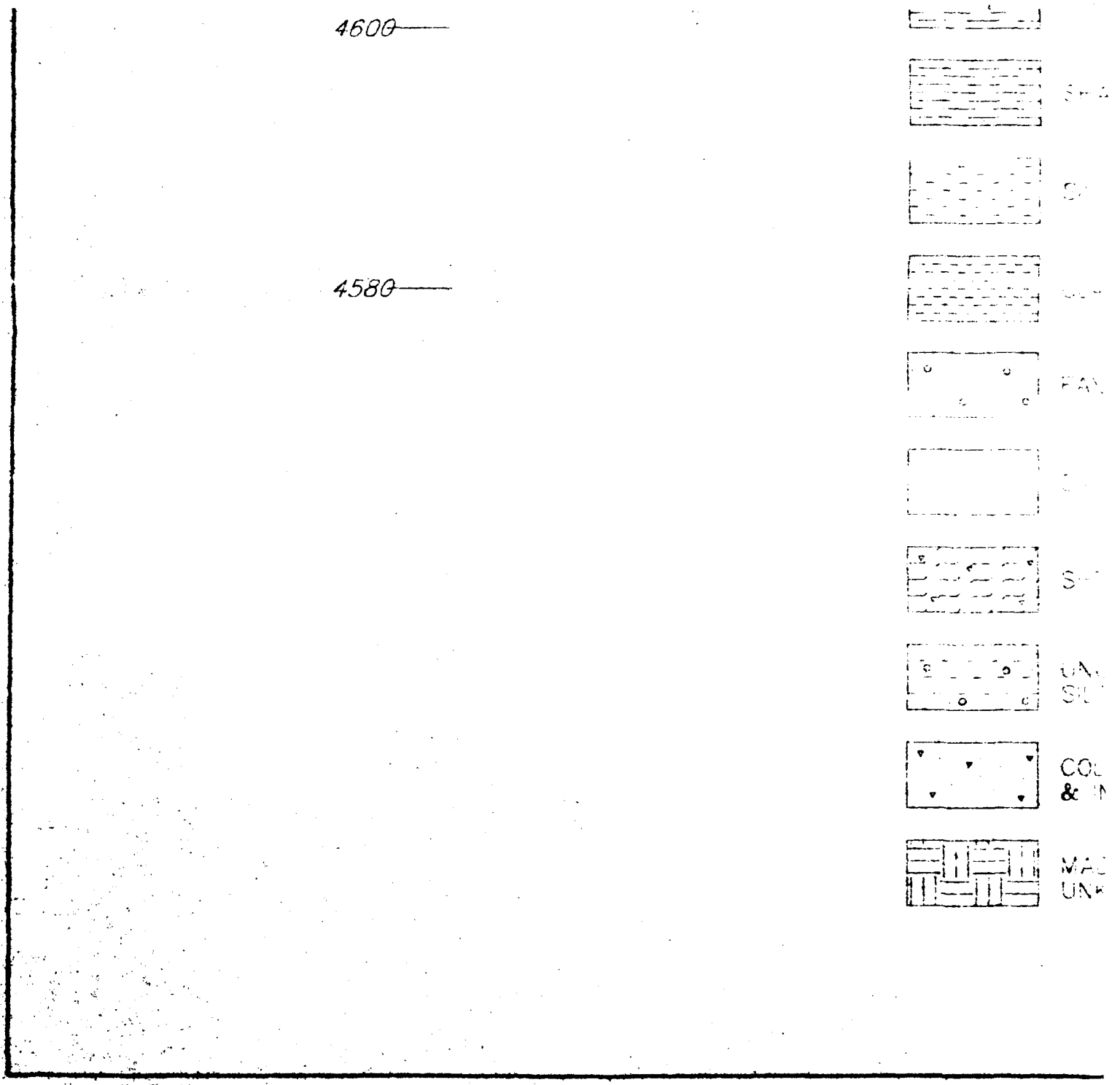
- DENTAL CONCRETE
- LIMESTONE
- ARGILLACEOUS LIMESTONE
- SHALE / LIMESTONE
- LIMESTONE & SHALE
- LIMEY SHALE
- SHALE
- SHALEY SILTSTONE

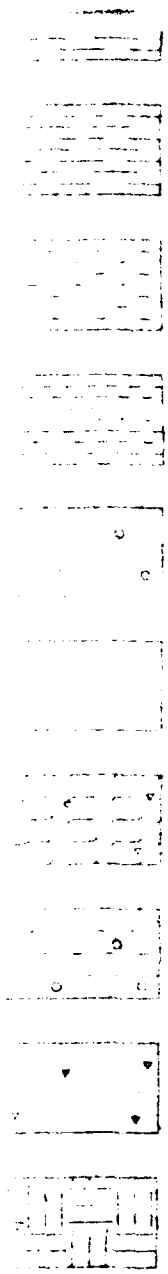

ALLUVIUM
&
SLOPEWASH


2
2
5
MADERA
LIMESTONE
SANTA FE
GROUP



MATCHLINE B–B'


4600
4580



SHALEY SILTSTONE

SILTSTONE

CLAYSTONE

FANGLOMERATE / CONGLOMERATE

SANDSTONE

SHEAR / BRECCIA

UNCONSOLIDATED ALLUVIUM
SILT, SAND, GRAVEL

COLLAPSE BRECCIA
& INFILLING MATL.

MADERA LIMESTONE
UNKNOWN LITHOLOGY

CONTACT BETWEEN MAPPE
UNITS, DASHED WHERE AC

FAULTS SHOWING RE
MENTS, DASHED WHERE A

FINAL EXCAVATED SURFAC

① MAPPED UNIT–CORRESPON
WITH NUMBER IN TABLE C

⑤ MAPPED UNIT–CORRESPON
WITH NUMBER IN TABLE C

EN MAPPED ROCK
WHERE ASSUMED

RELATIVE MOVE-
WHERE ASSUMED

D SURFACE

ORRESPONDS
TABLE C-1

ORRESPONDS
TABLE C-2

SCALE IN FEET:



U.

DESIGNED BY: C.B.D.

DRAWN BY: S.D.M.

CHECKED BY: W.S.B.

RIO GRAN

G

David E. Wa

ENGINEER

DATE: 10/20/82



U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

D BY:
I.D.
BY:
).M.
) BY:
S.B.

RIO GRANDE & TRIBUTARIES — TRUTH OR CONSEQUENCES, N.M.

CUCHILLO NEGRO DAM

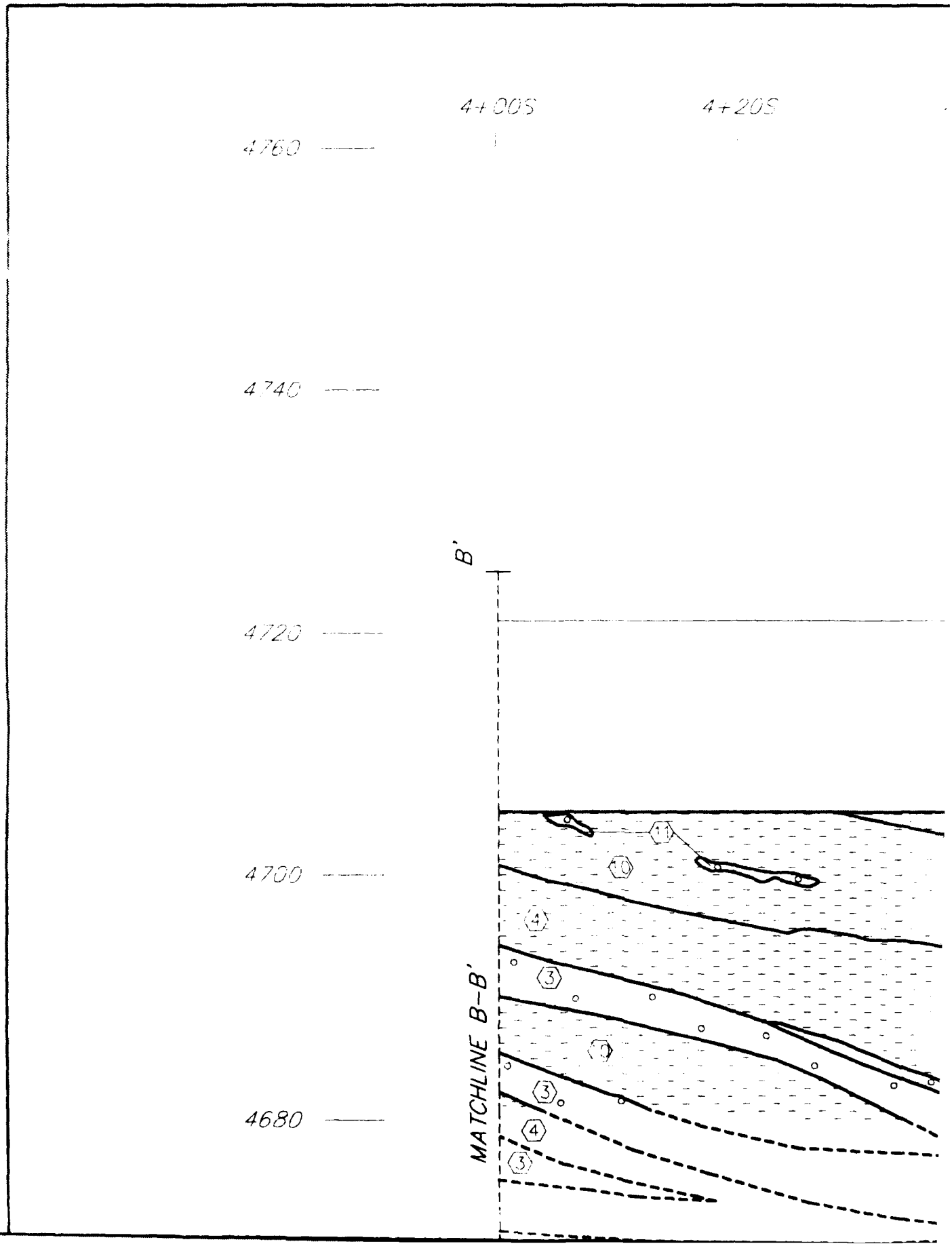
FOUNDATION REPORT

GEOLOGIC CROSS SECTION ALONG AUXILIARY SPILLWAY SILL AXIS LOOKING S14°E

David E. Wright
NEER
10/20/82

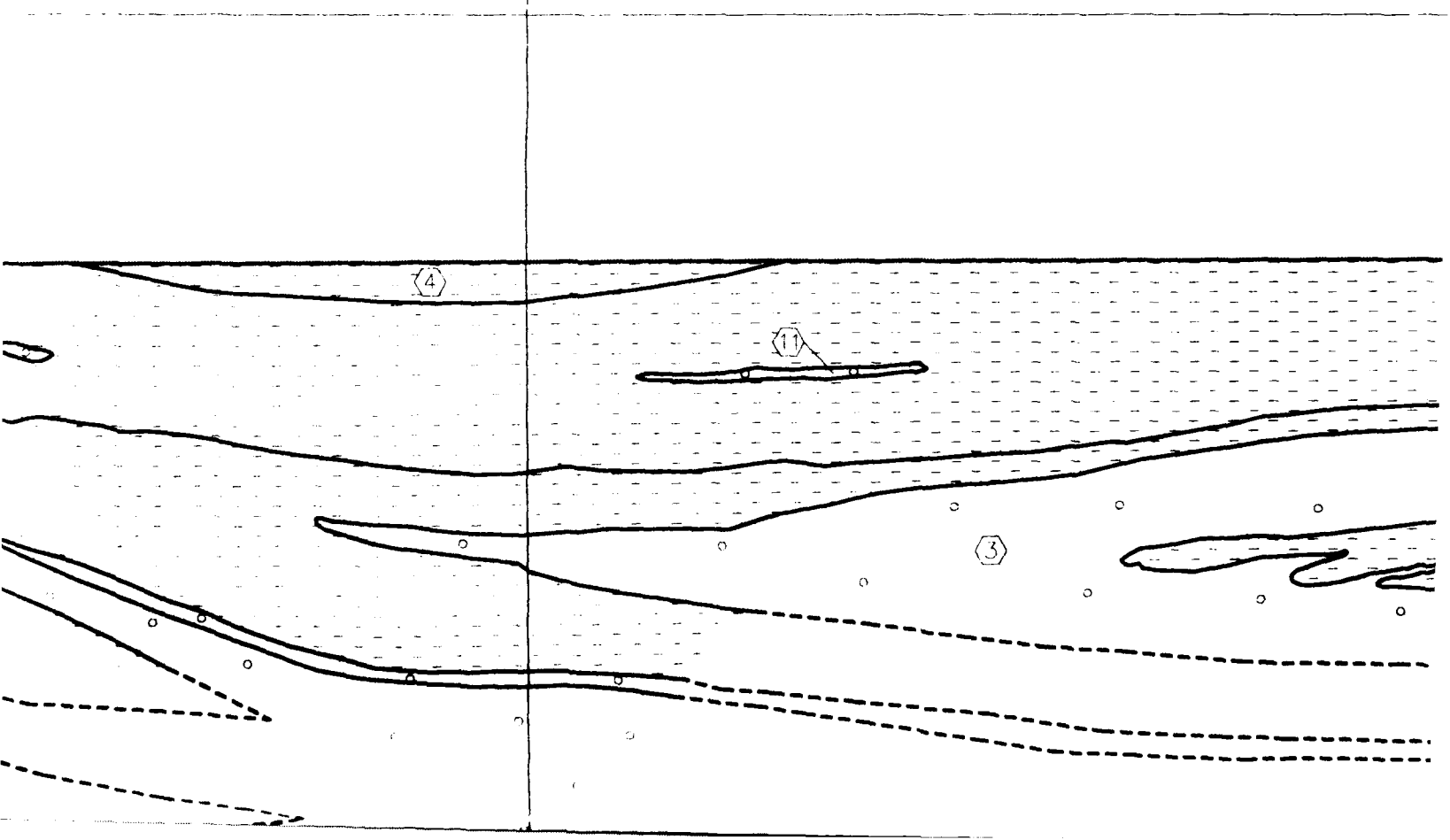
FILE NUMBER

PLATE
A-9


4+00S
4+20S
4760
4740
4720
4700
4680
B'
MATCHLINE B-B'
1
10
4
3
3
3
4
3

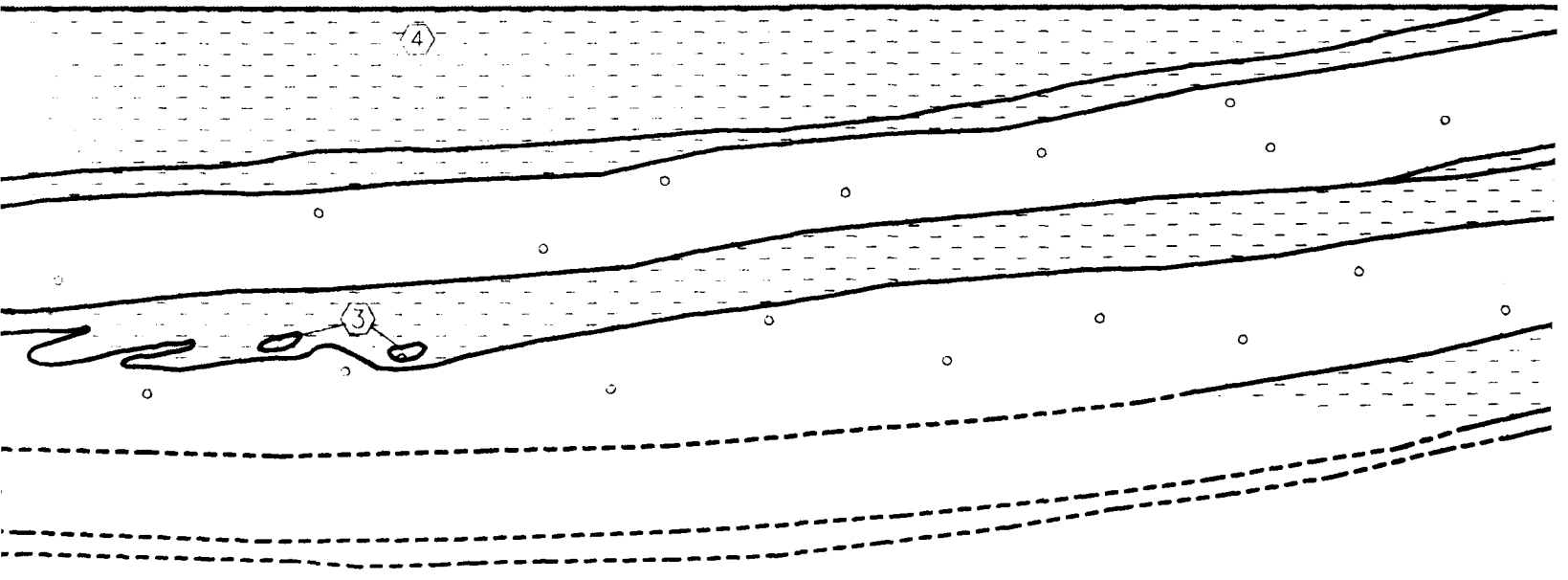
4+40S 4+60S 4+80S 5+00S

CN-CH-45



5+20S 5+40S 5+60S 5+80S

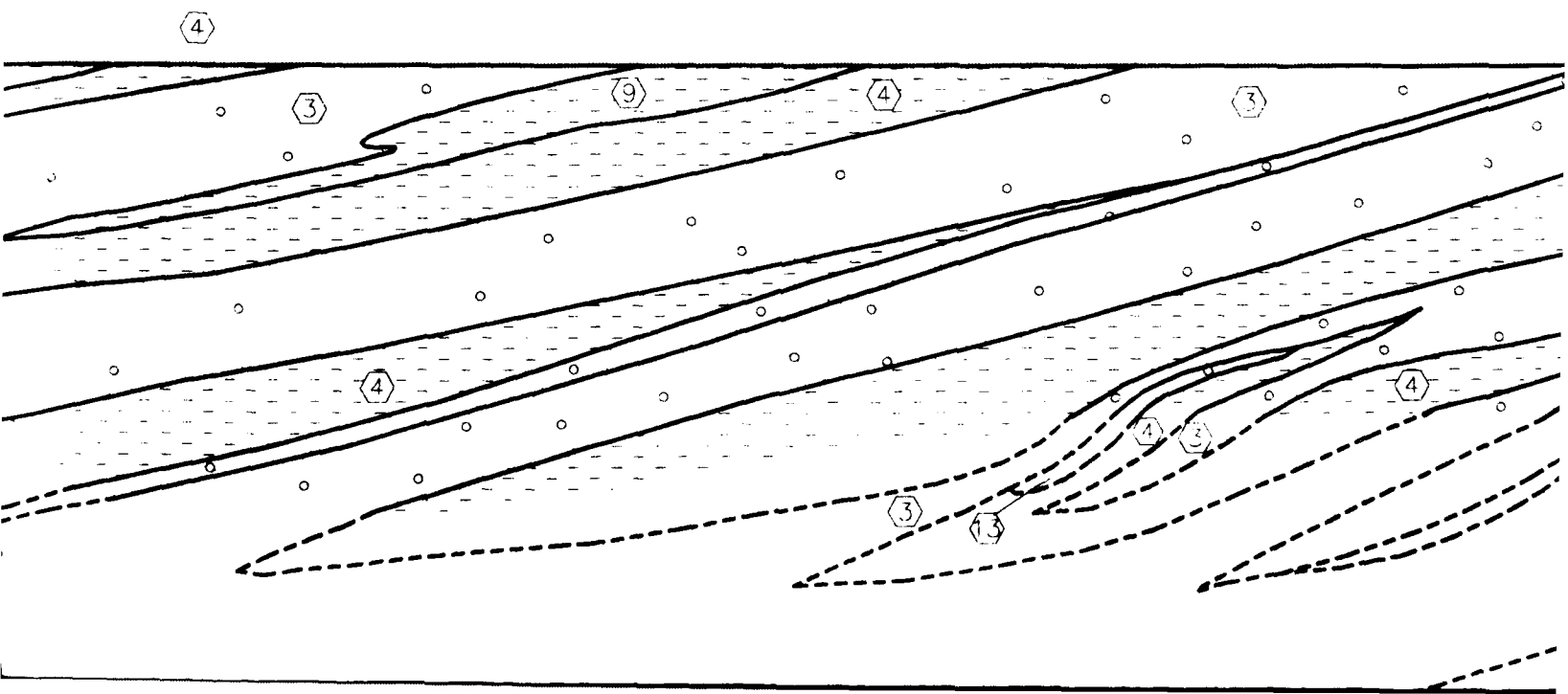
AUXILIARY SPILLWAY OGEE CREST

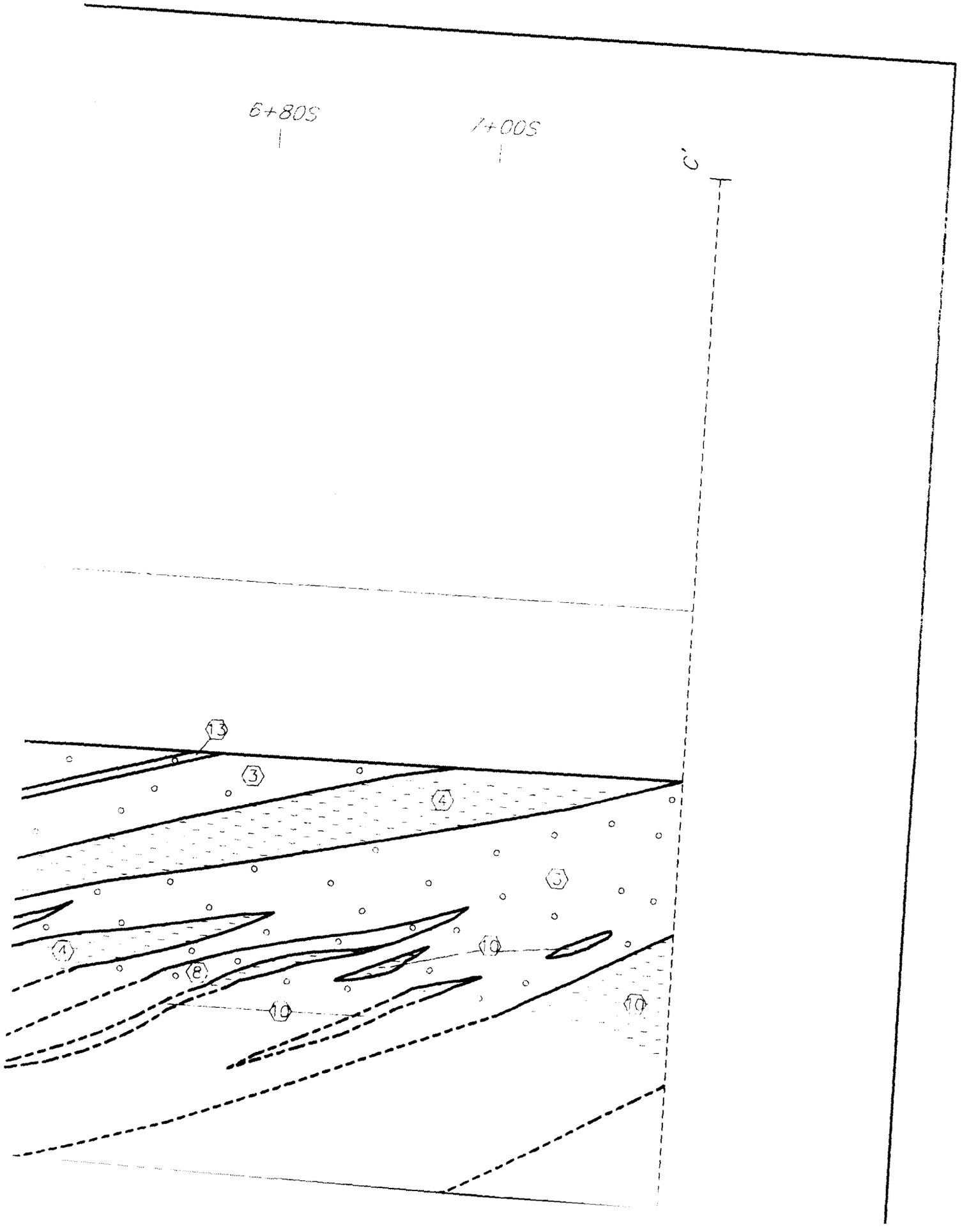


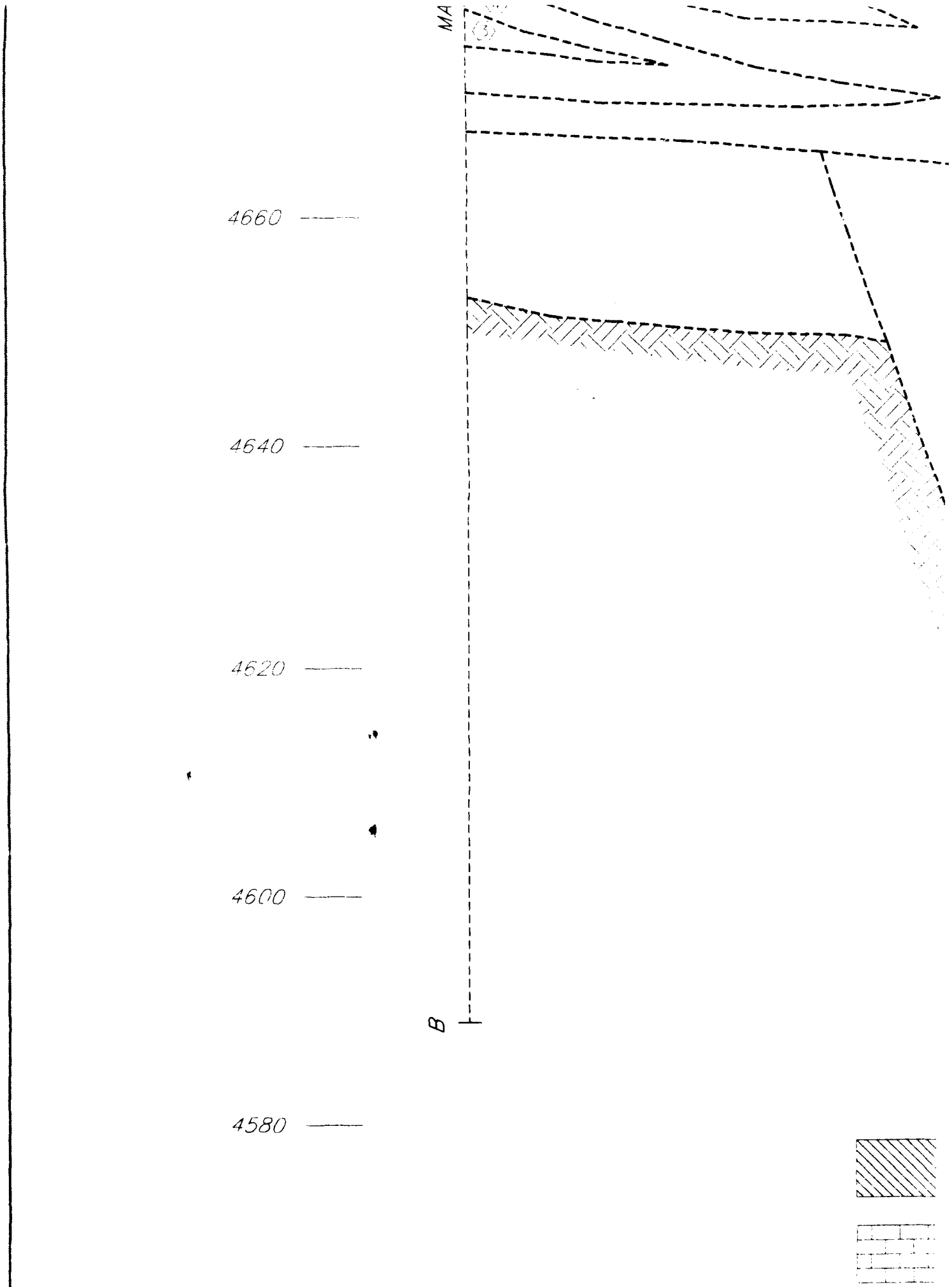
6+00S

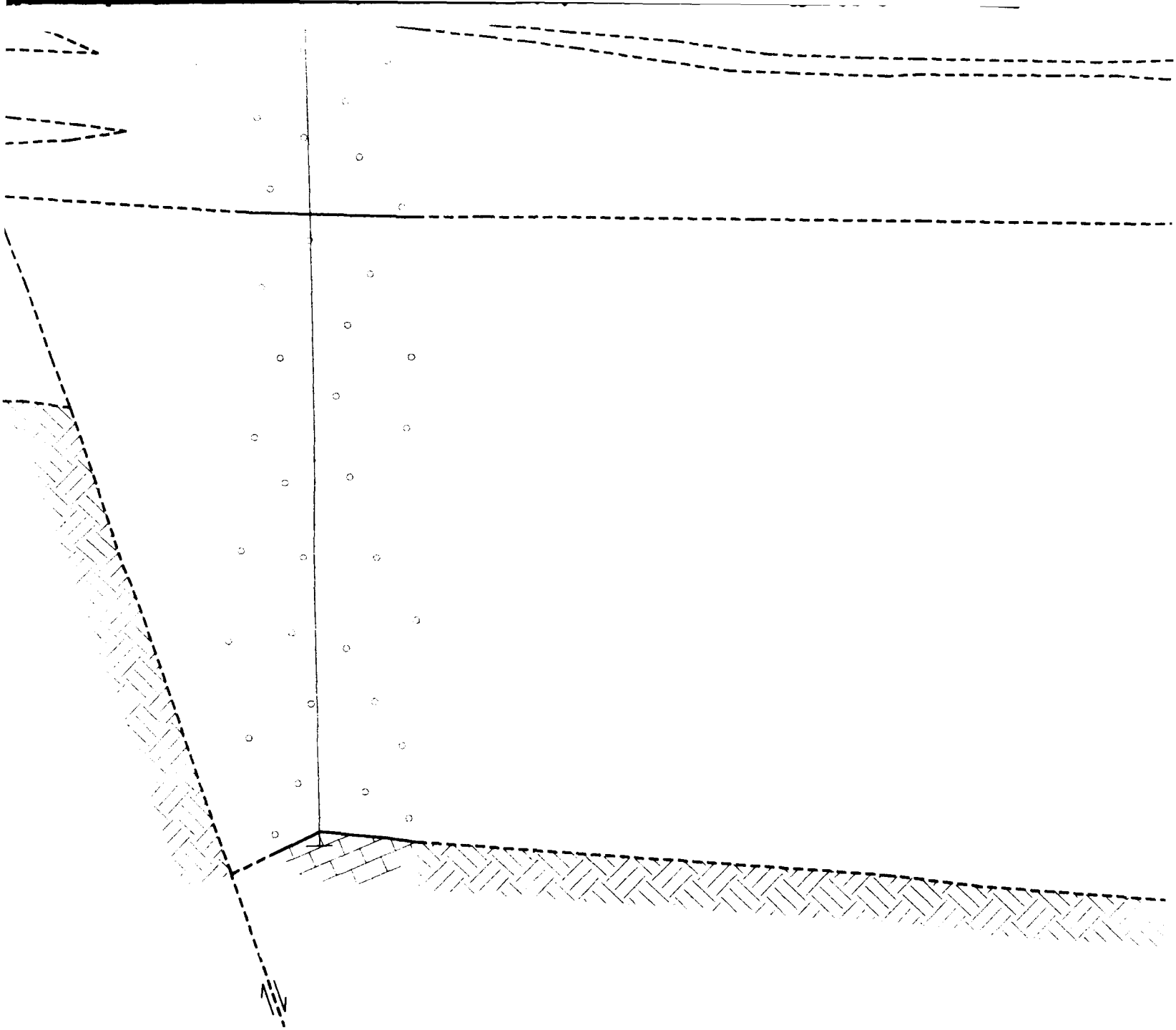
6+20S

6+40S




6+80S
1+00S
C'
13
3
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10


MA
B
4660
4640
4620
4600
4580

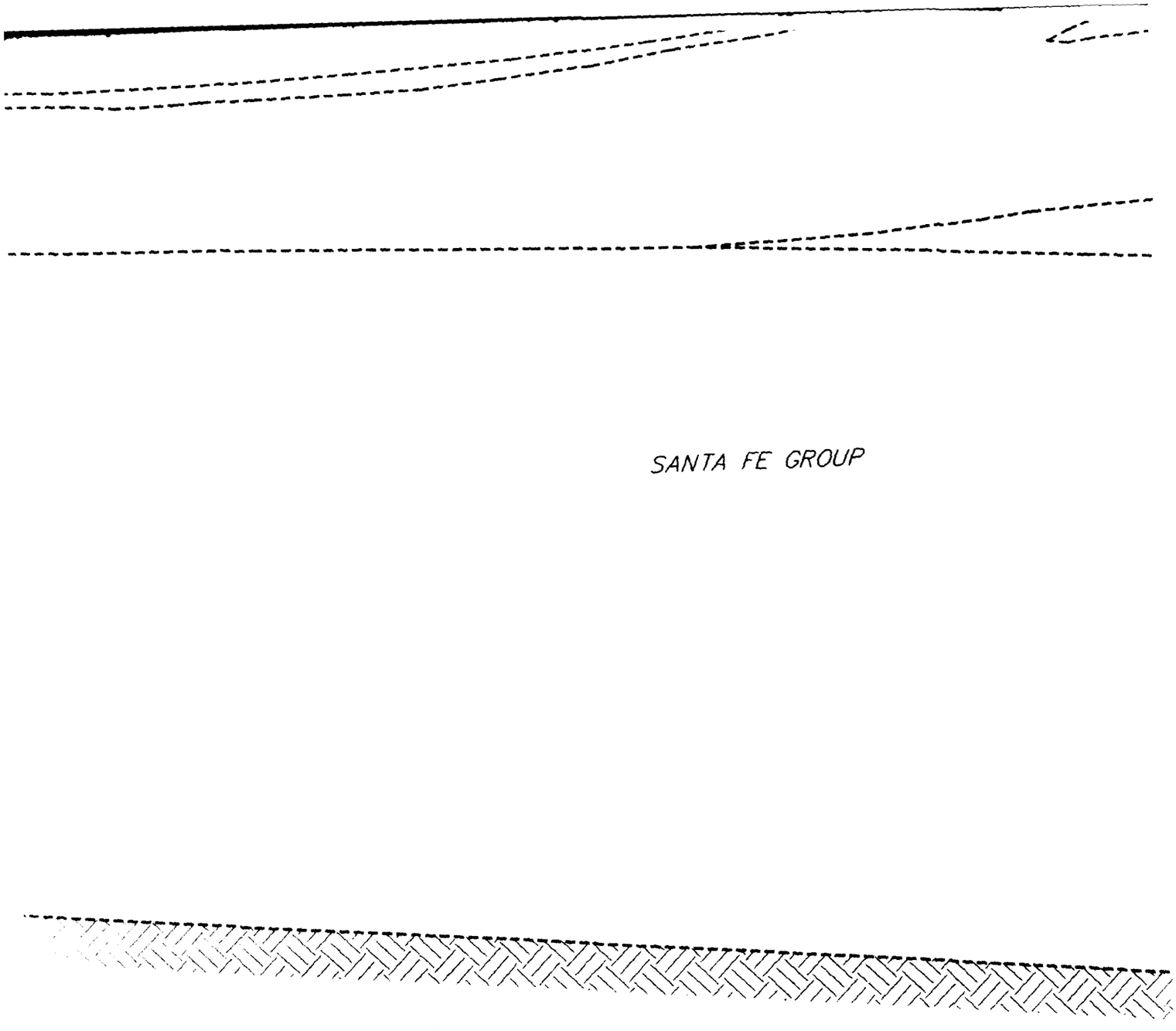


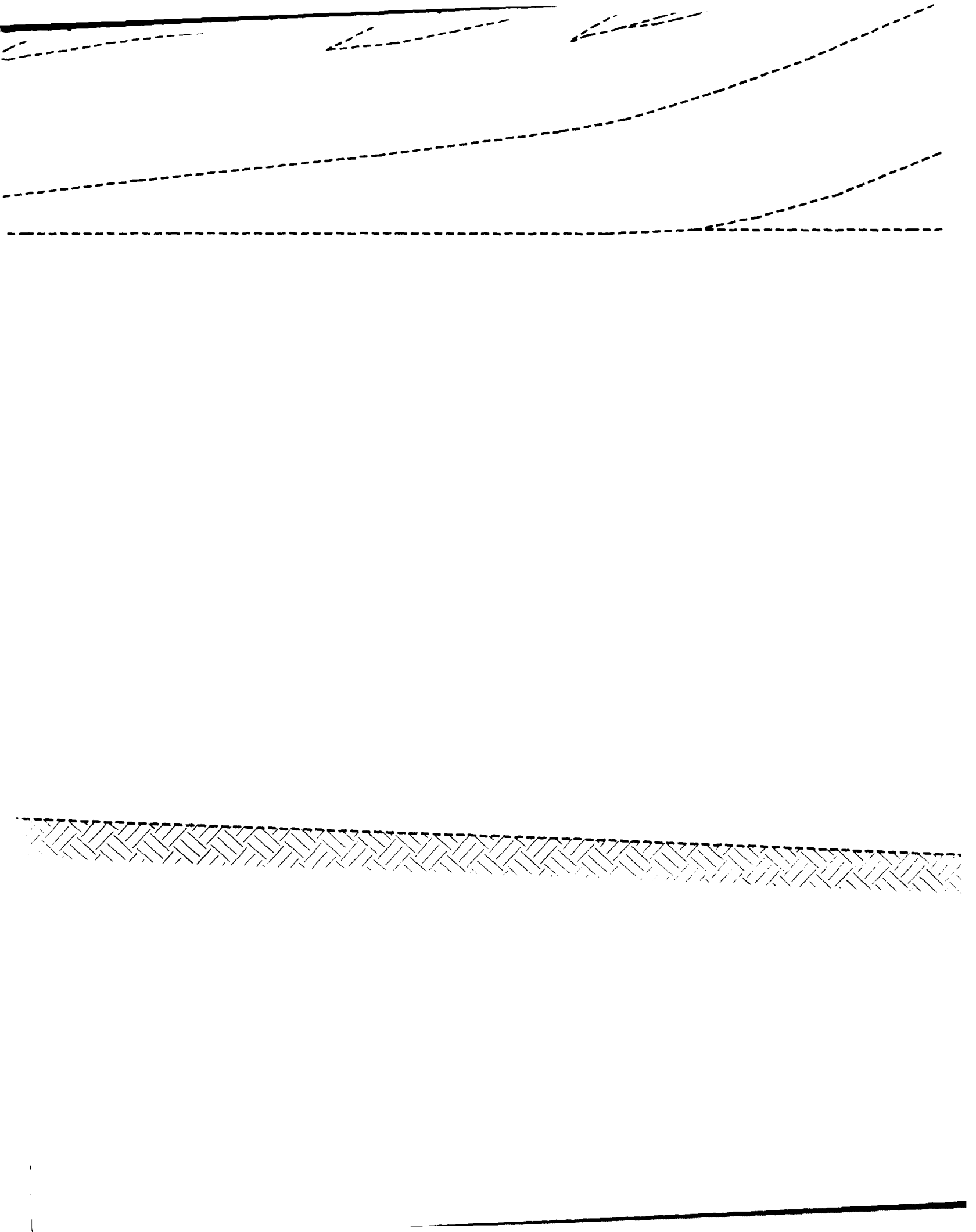
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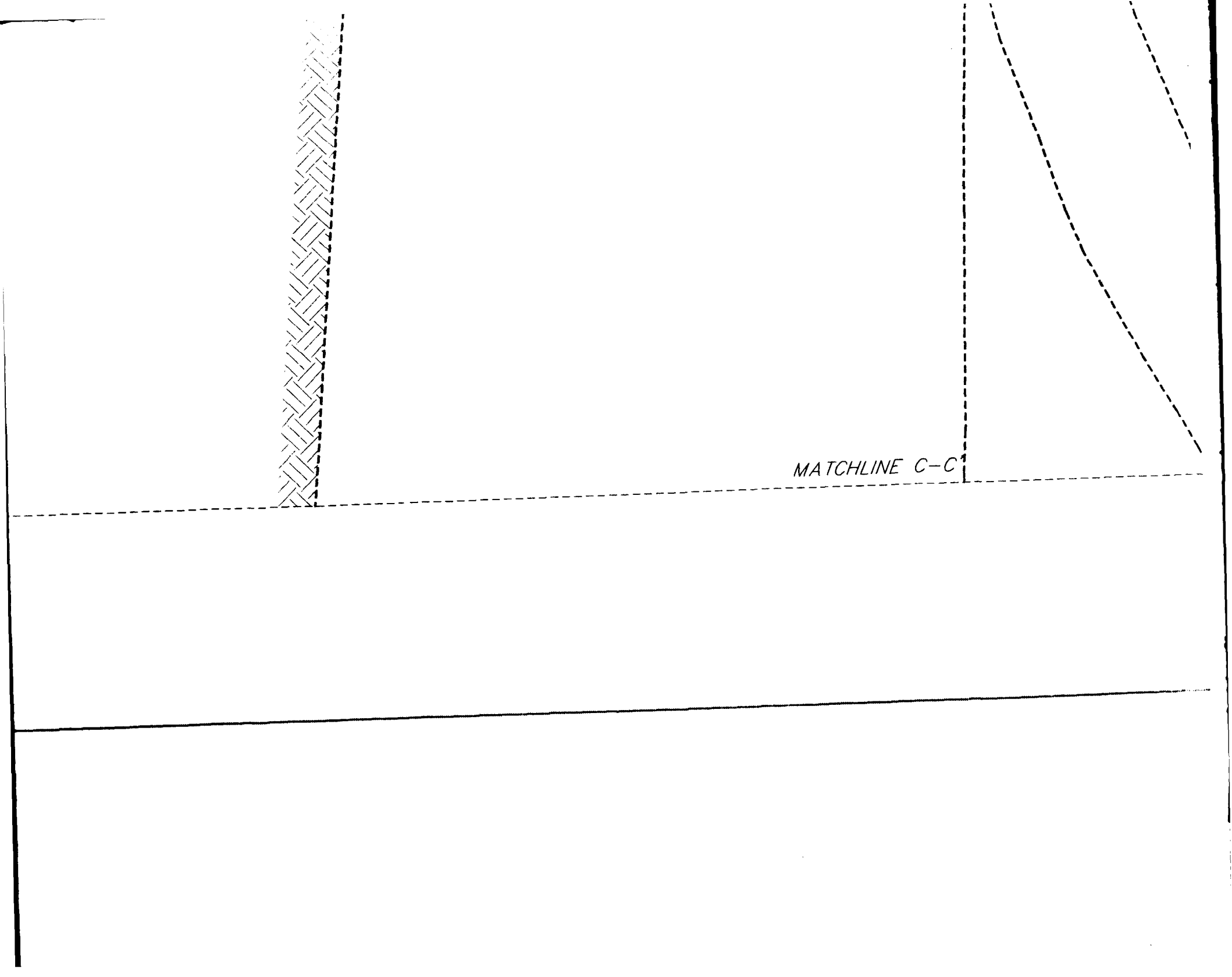
DENTAL CONCRETE

CLAYSTONE

ANGLOMERATE / CONGLOM


SANTA FE GROUP
MADERA LIMESTONE



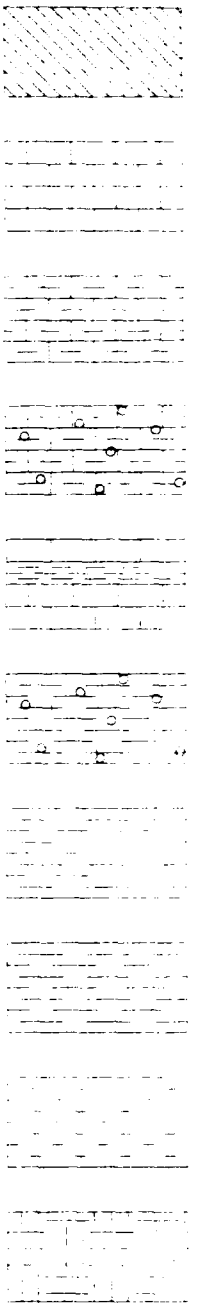

MATCHLINE C–C'

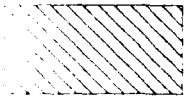
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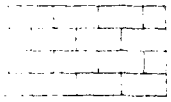
4560

4540

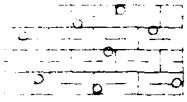
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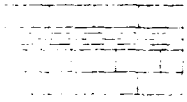



DENTAL CONCRETE


LIMESTONE


ARGILLACEOUS LIMESTONE


SHALE / LIMESTONE


LIMESTONE & SHALE


LIMEY SHALE


SHALE


SHALEY SILTSTONE

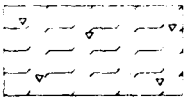

SILTSTONE


MADERA LIMESTONE


CLAYSTONE


CONGLOMERATE / CONGLOM


SANDSTONE


SHEAR / BRECCIA


UNCONSOLIDATED ALLUVIUM
SILT, SAND, GRAVEL


COLLAPSE BRECCIA
& INFILLING MATL

CONTACT BETWEEN MAPP
UNITS, DASHED WHERE


FAULTS SHOWING RELATIVE
MENTS, DASHED WHERE ASS

FINAL EXCAVATED SURFACE



MAPPED UNIT-CORRESPOND
WITH NUMBER IN TABLE



MAPPED UNIT CORRESPO
WITH NUMBER IN TABLE


SCALE IN FEET:
10 5 0 10 20

U.

DESIGNED BY: C.B.D.	RIO GRANDE
DRAWN BY: S.D.M.	GE
CHECKED BY: W.S.B.	

ENGINEER

DATE: Sept. 1



U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

ED BY:
.3 D.

BY:
C.M.

ED BY:
V S B

RIO GRANDE & TRIBUTARIES — TRUTH OR CONSEQUENCES, N.M.

CUCHILLO NEGRO DAM

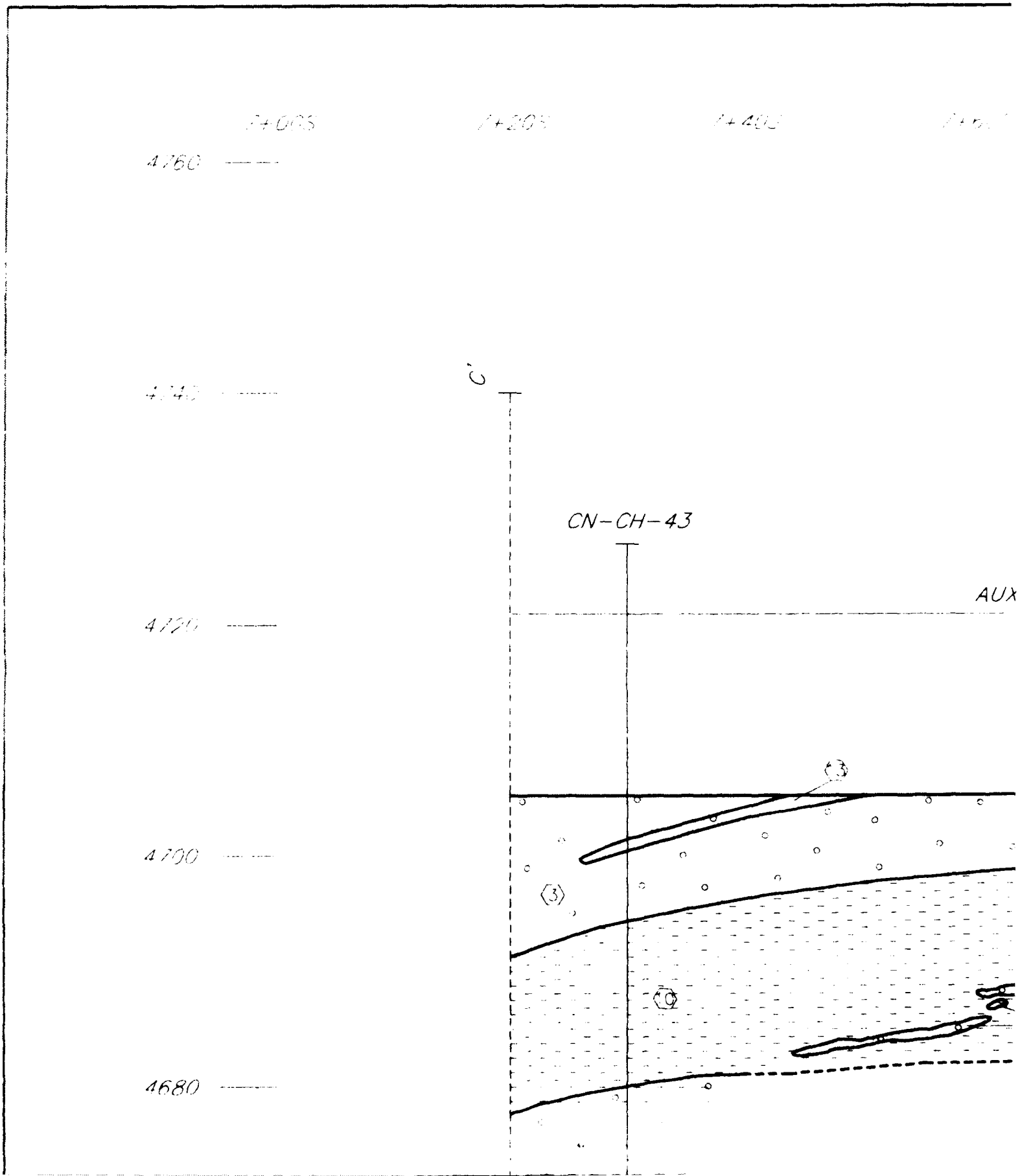
FOUNDATION REPORT

GEOLOGIC CROSS SECTION ALONG AUXILIARY SPILLWAY SILL AXIS LOOKING S14°E

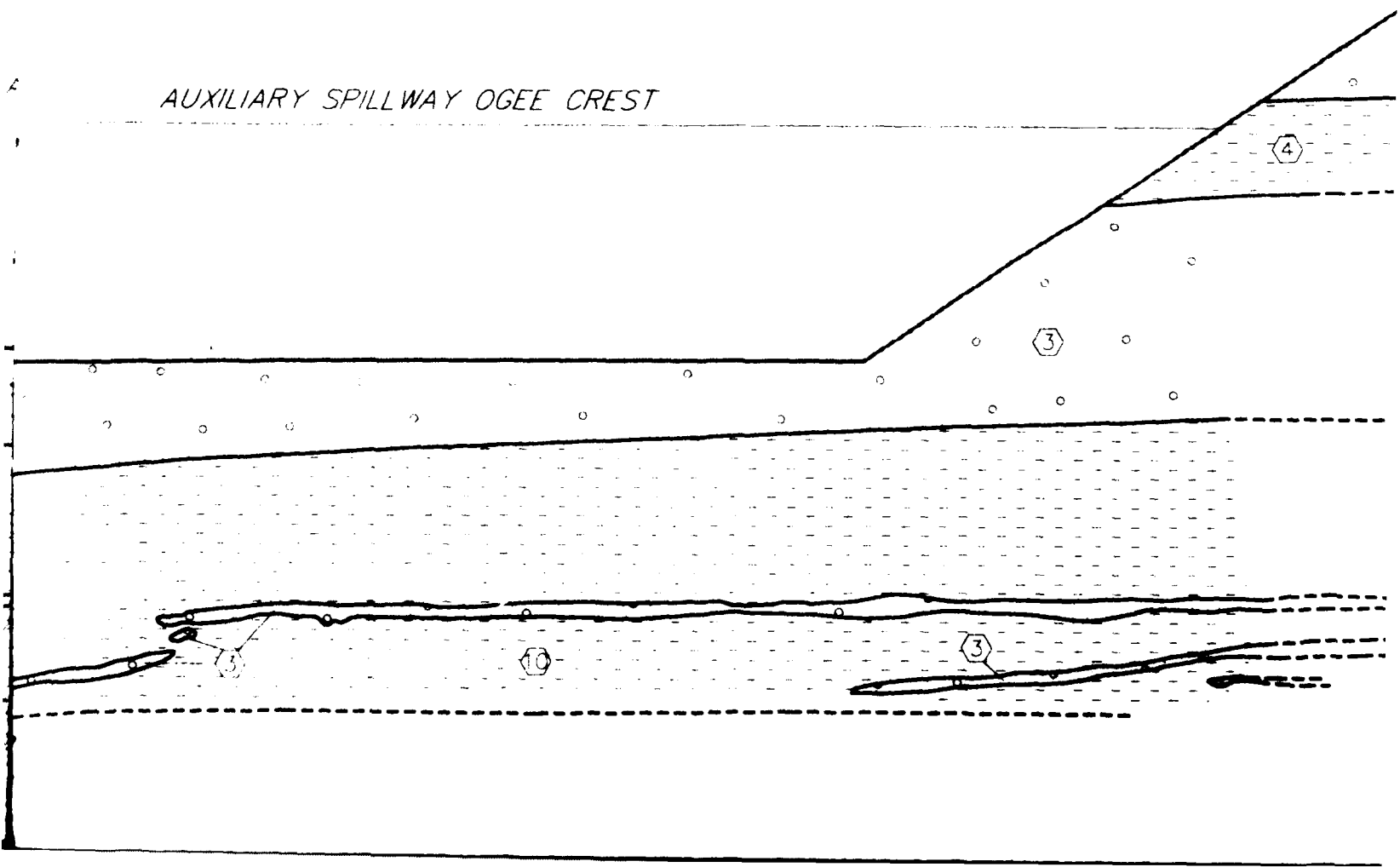
David E. Wright
GINEER
Sect. 12

FILE NUMBER

PLATE
A-10


4760
4740
4720
4700
4680
C'
CN-CH-43
AUX
3
3
2

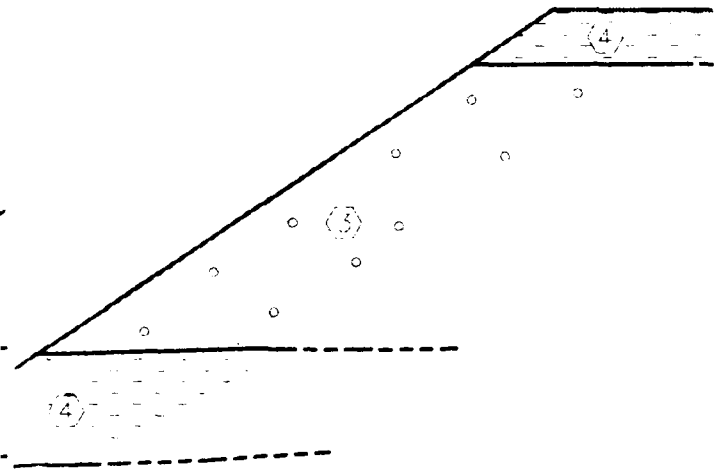
7+80S 7+80S 8+00S 8+20S 8+4

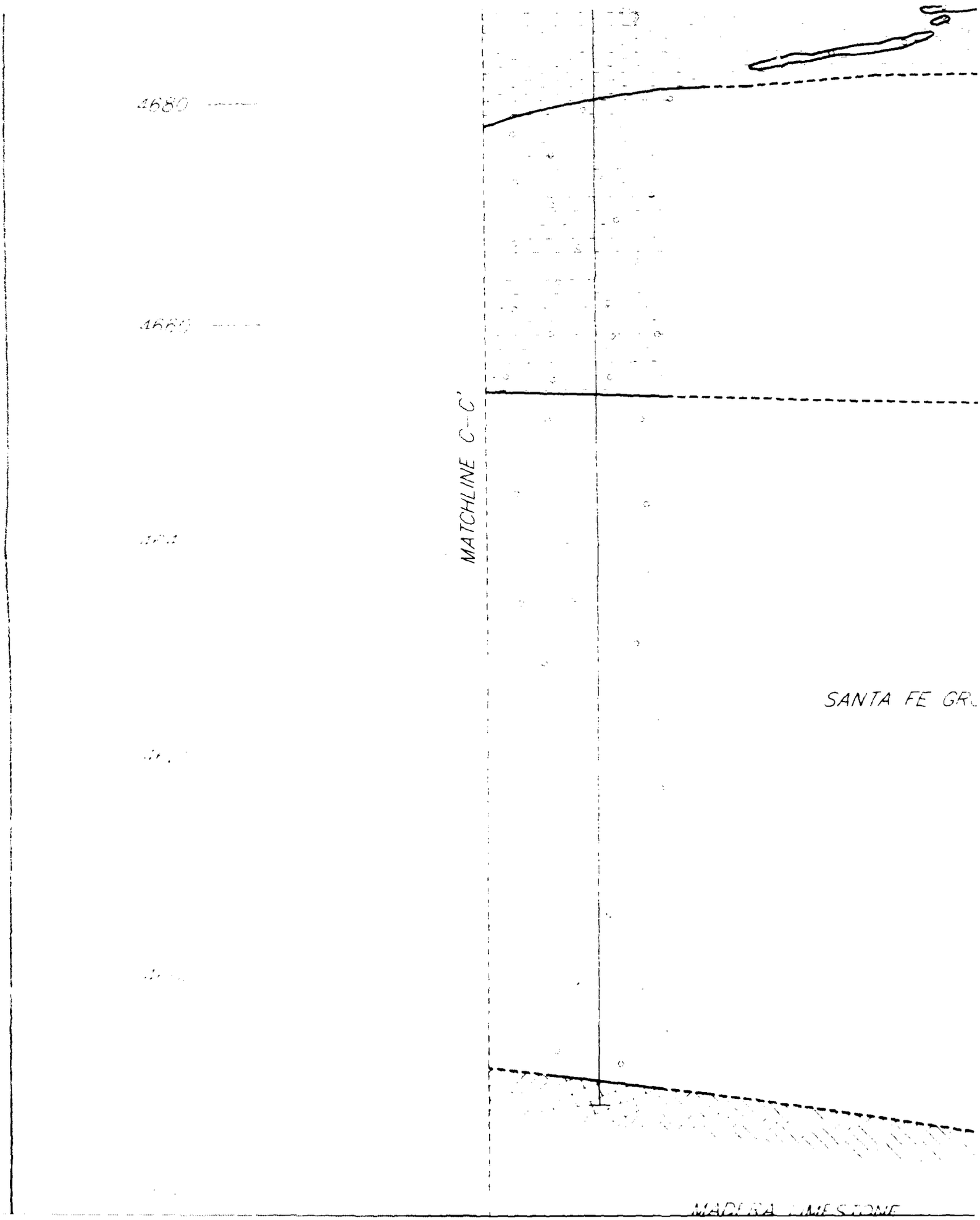


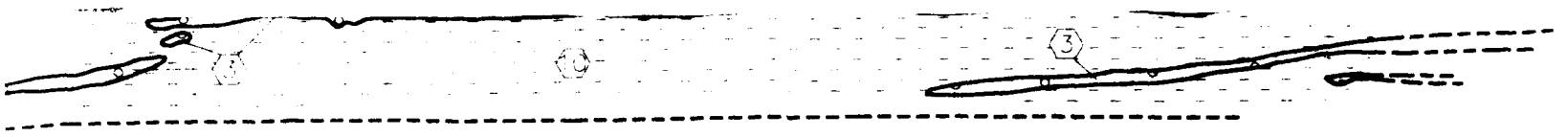
8+405

8+815

9+00



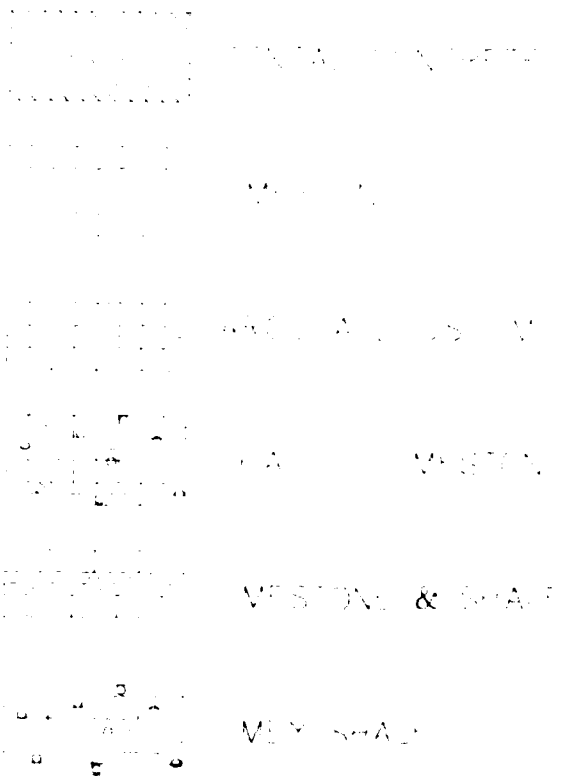

4680
MATCHLINE C–C'
SANTA FE GR
MADERA LIMESTONE



SANTA FE GROUP

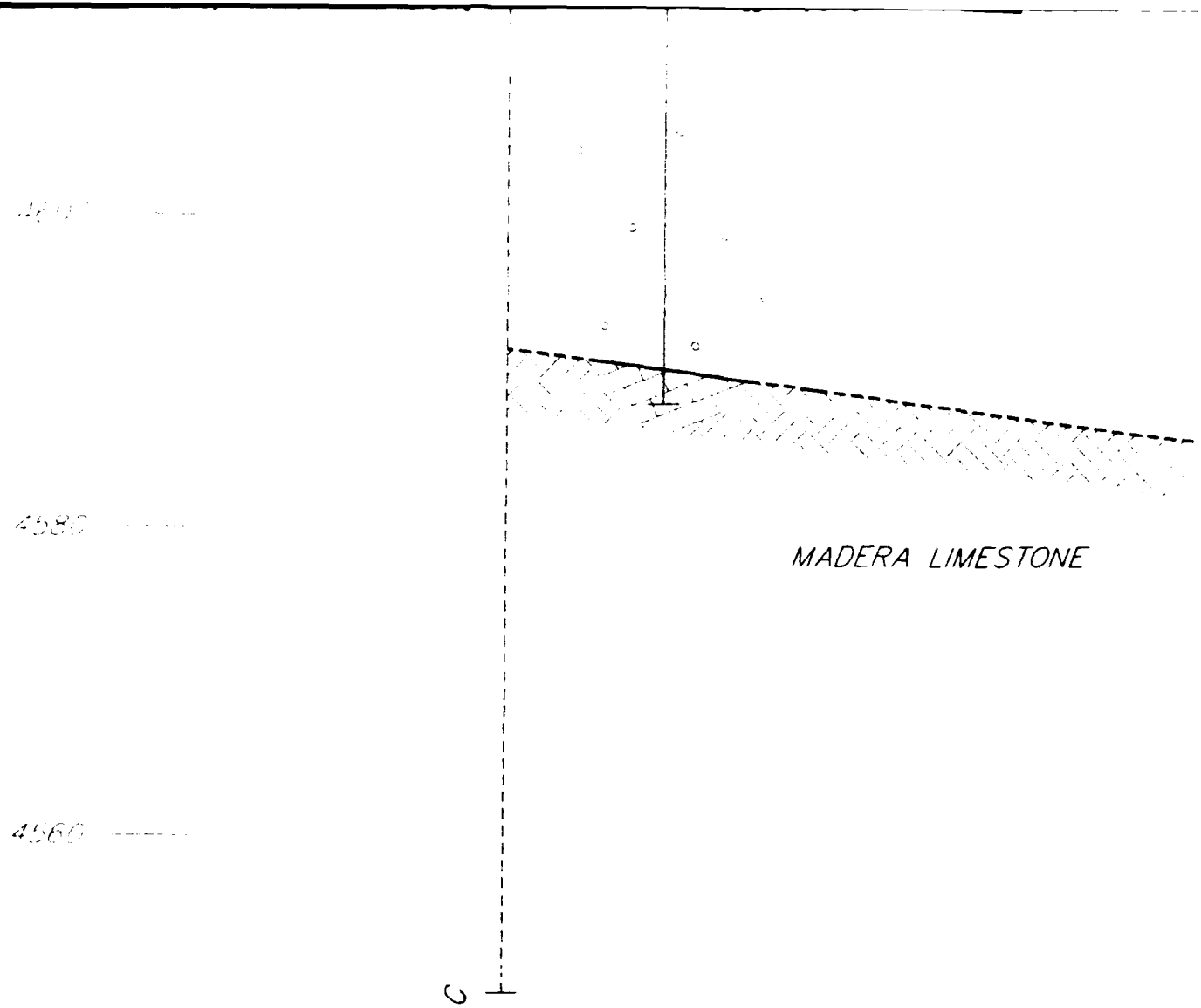
TOME

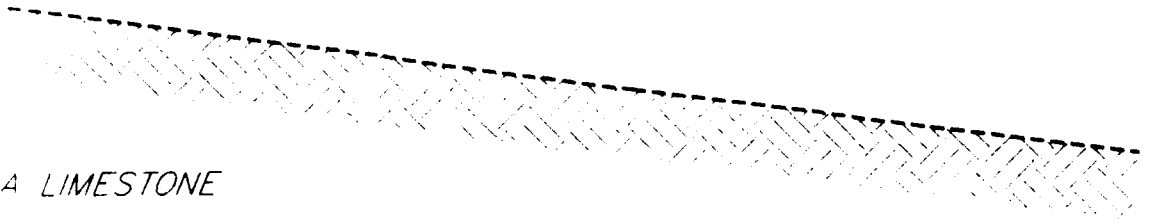
LEG



LEGEND:

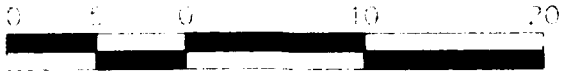

CLAYSTONE
SANDSTONE
UNCONSOLIDATED ALLUVIUM
SILT, SAND, GRAVEL
COLLAPSE BRECCIA
CONTACT BETWEEN MAPPED ROCK

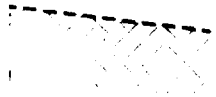

4580
MADERA LIMESTONE
4560
4540



A LIMESTONE

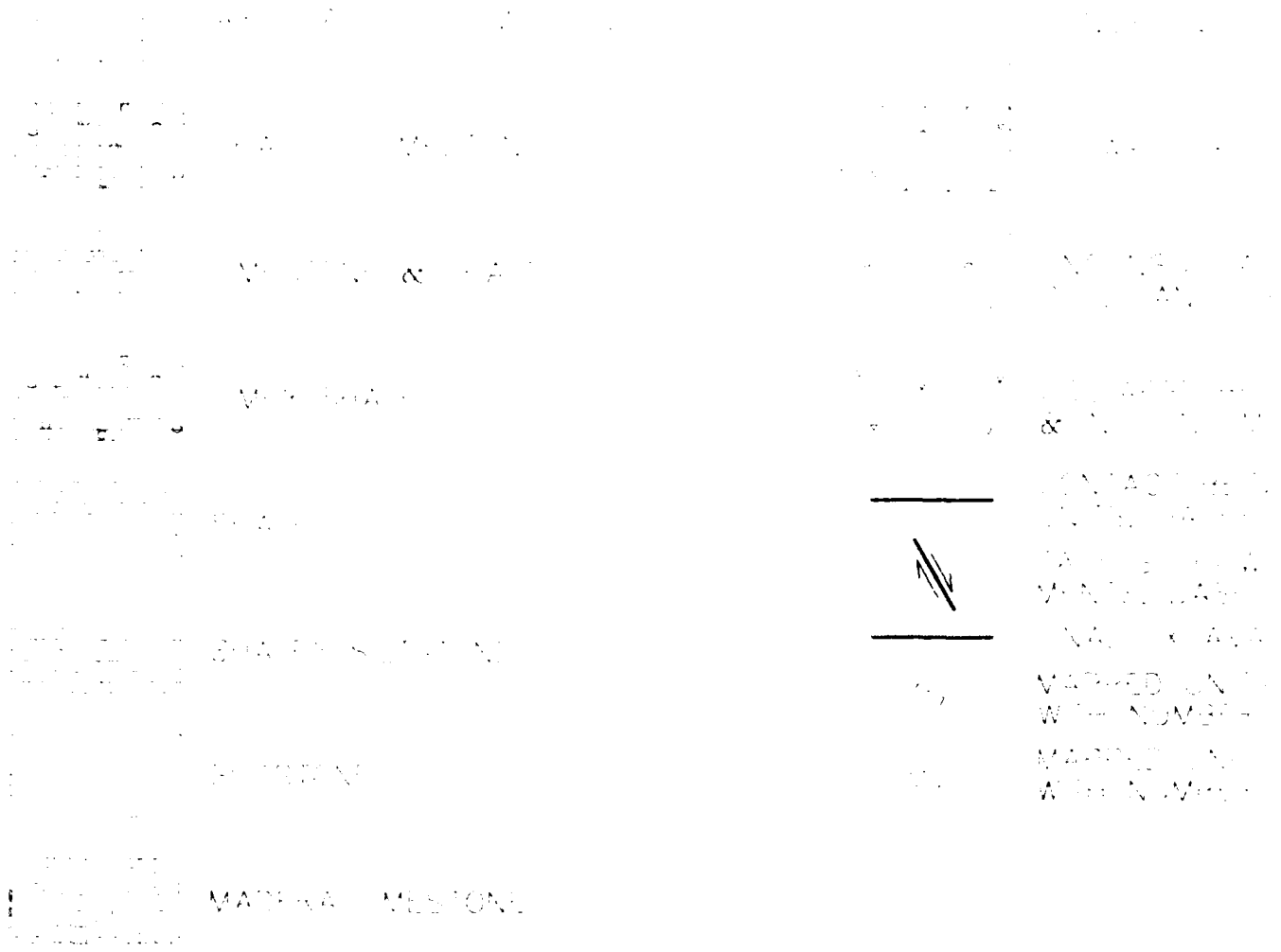
SCALE IN FEET:





E T:

20

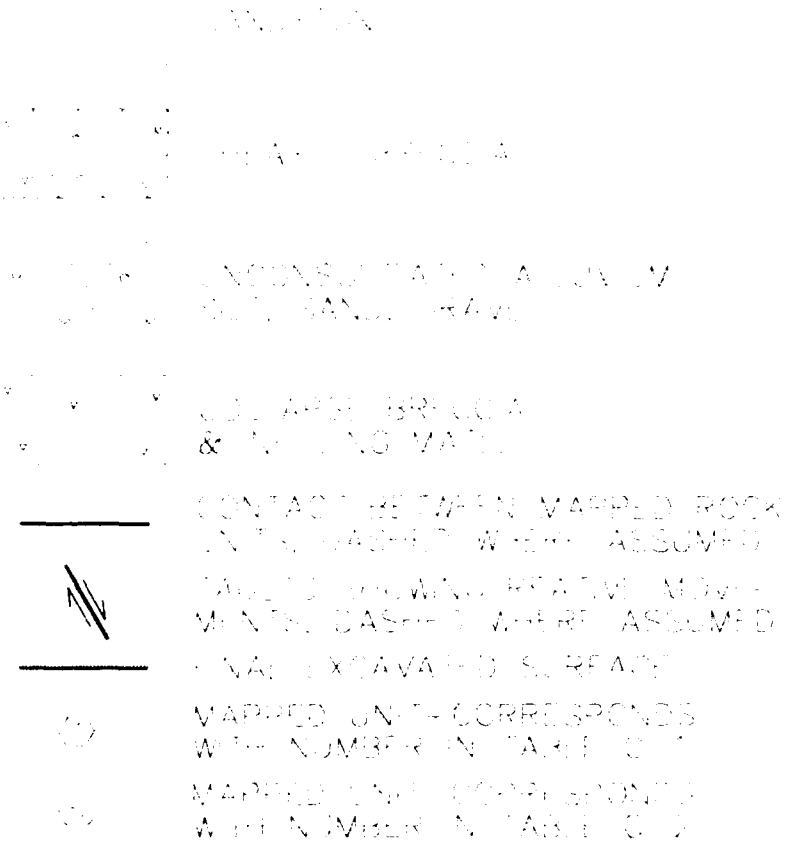


U. S. ARMY ENGINEER DISTRICT, ALE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

DESIGNED BY: C.B.D.	RIO GRANDE & TRIBUTARIES
DRAWN BY: S.D.M.	CUCHILLO NEGRO D
CHECKED BY: W.S.B.	FOUNDATION REPORT
	GEOLOGIC CROSS SEC AUXILIARY SPILLWAY LOOKING S14

David E. Wright
ENGINEER
DATE: 5 Oct. 92

FILE NUMBER



U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

RIO GRANDE & TRIBUTARIES — TRUTH OR CONSEQUENCES, N.M.

CUCHILLO NEGRO DAM

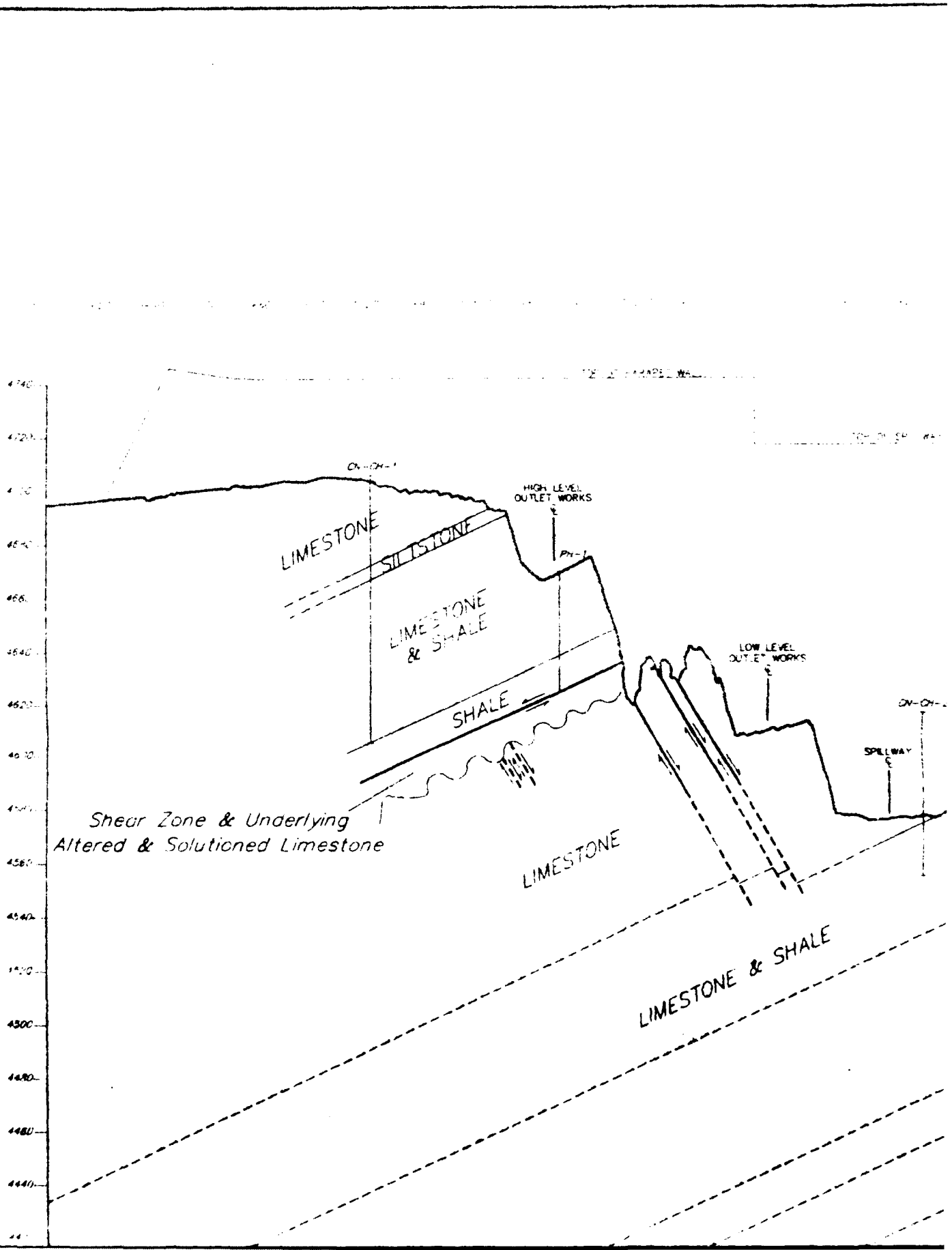
FOUNDATION REPORT

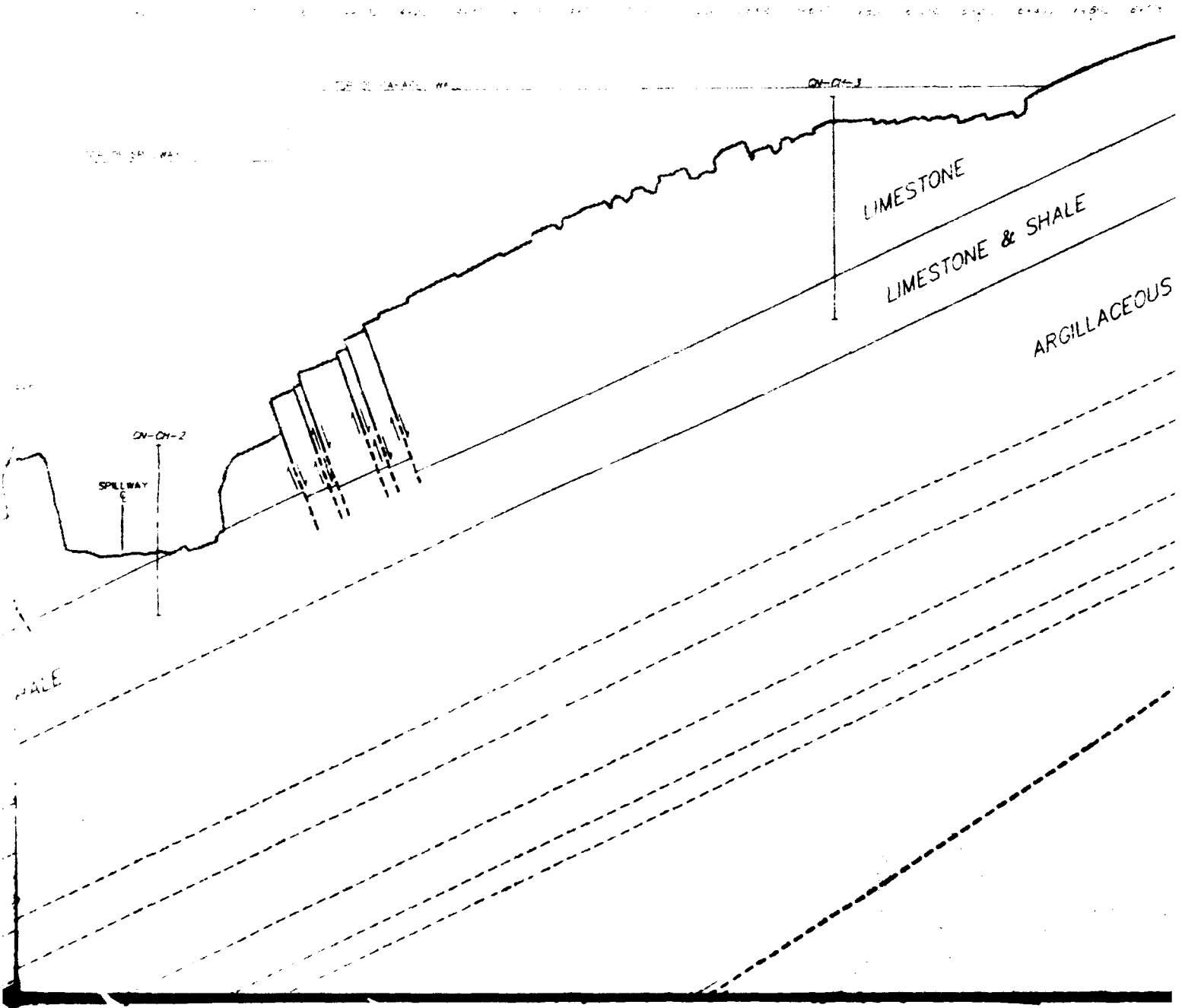
GEOLOGIC CROSS SECTION ALONG AUXILIARY SPILLWAY SILL AXIS LOOKING S14° E

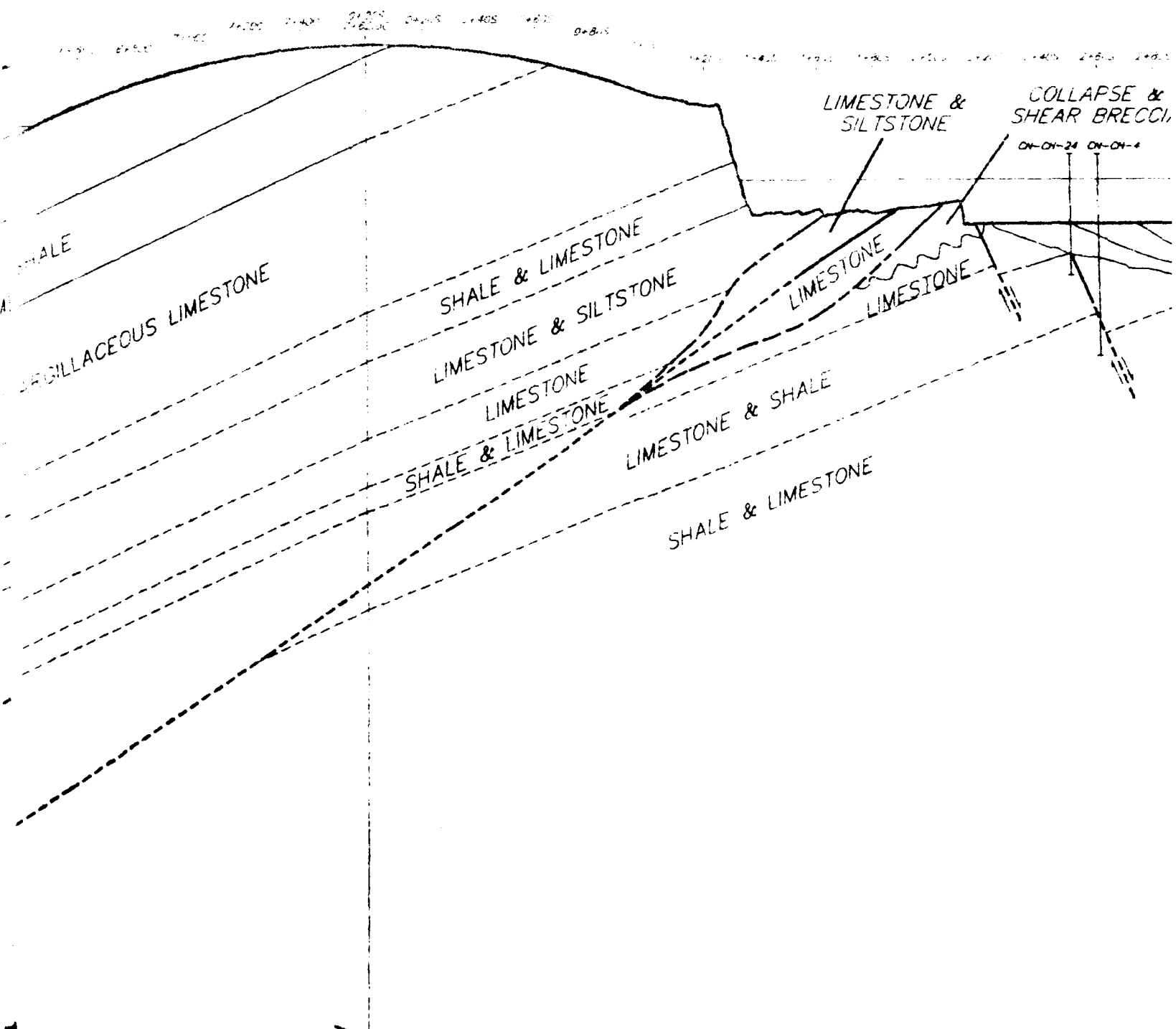
L. E. Wright
Sect. 9%

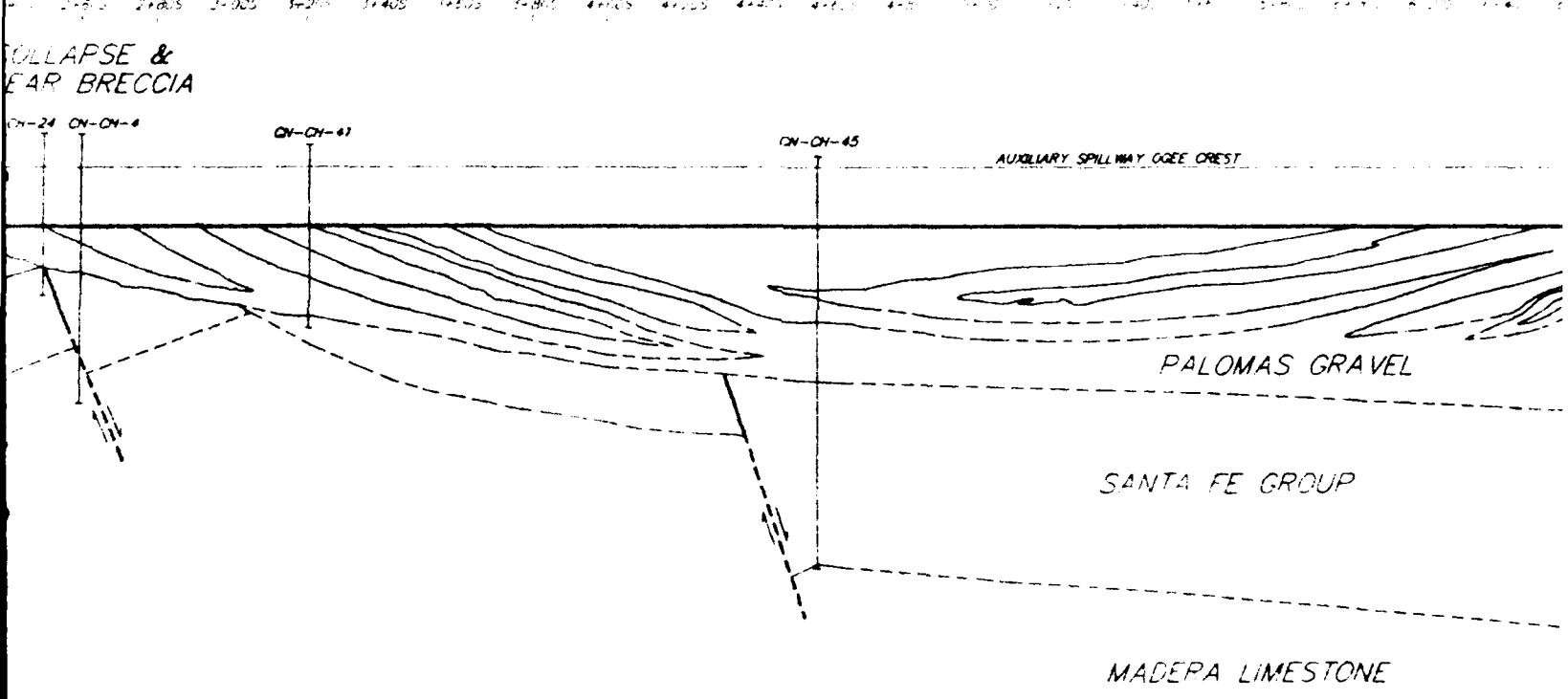
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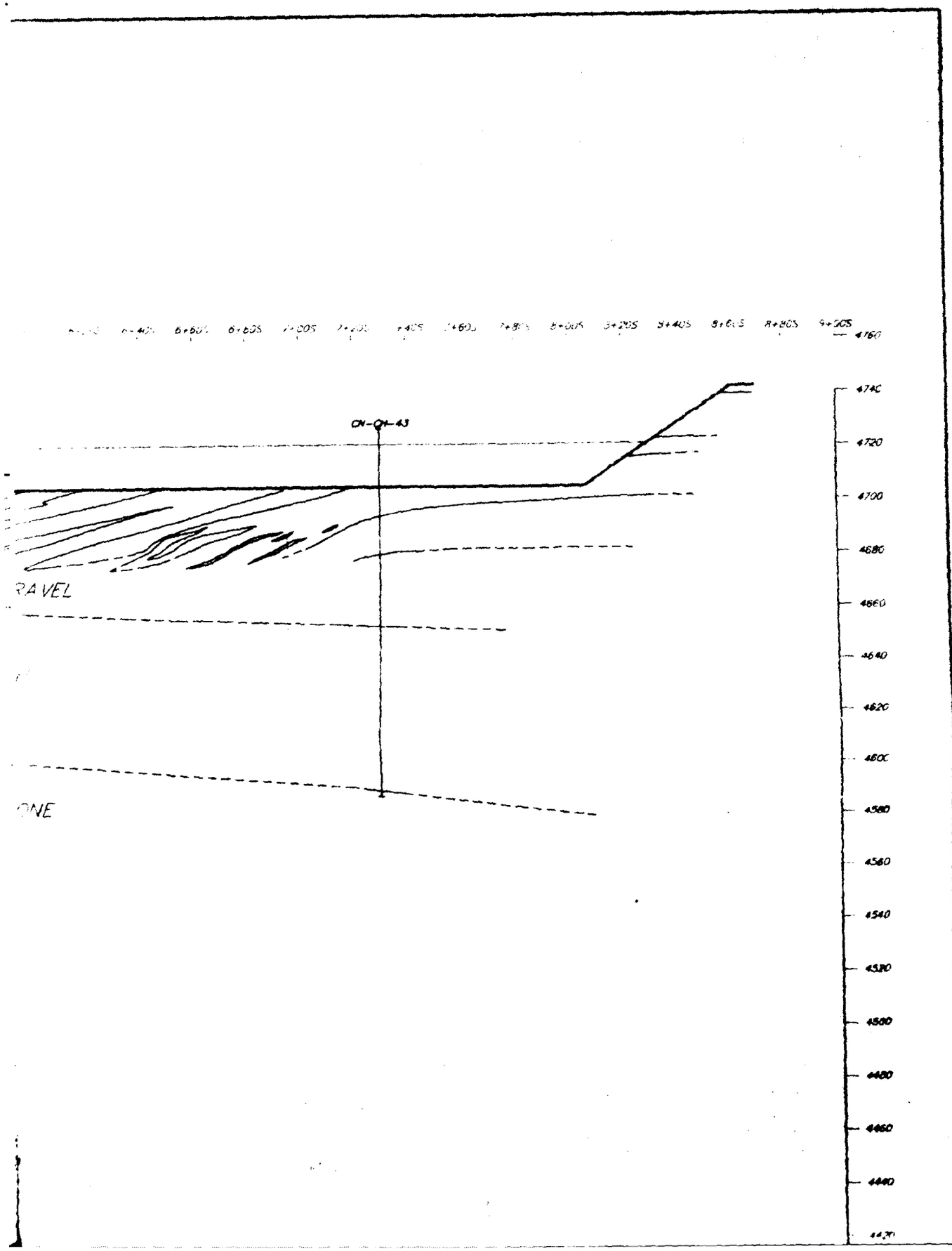
PLATE
A-11

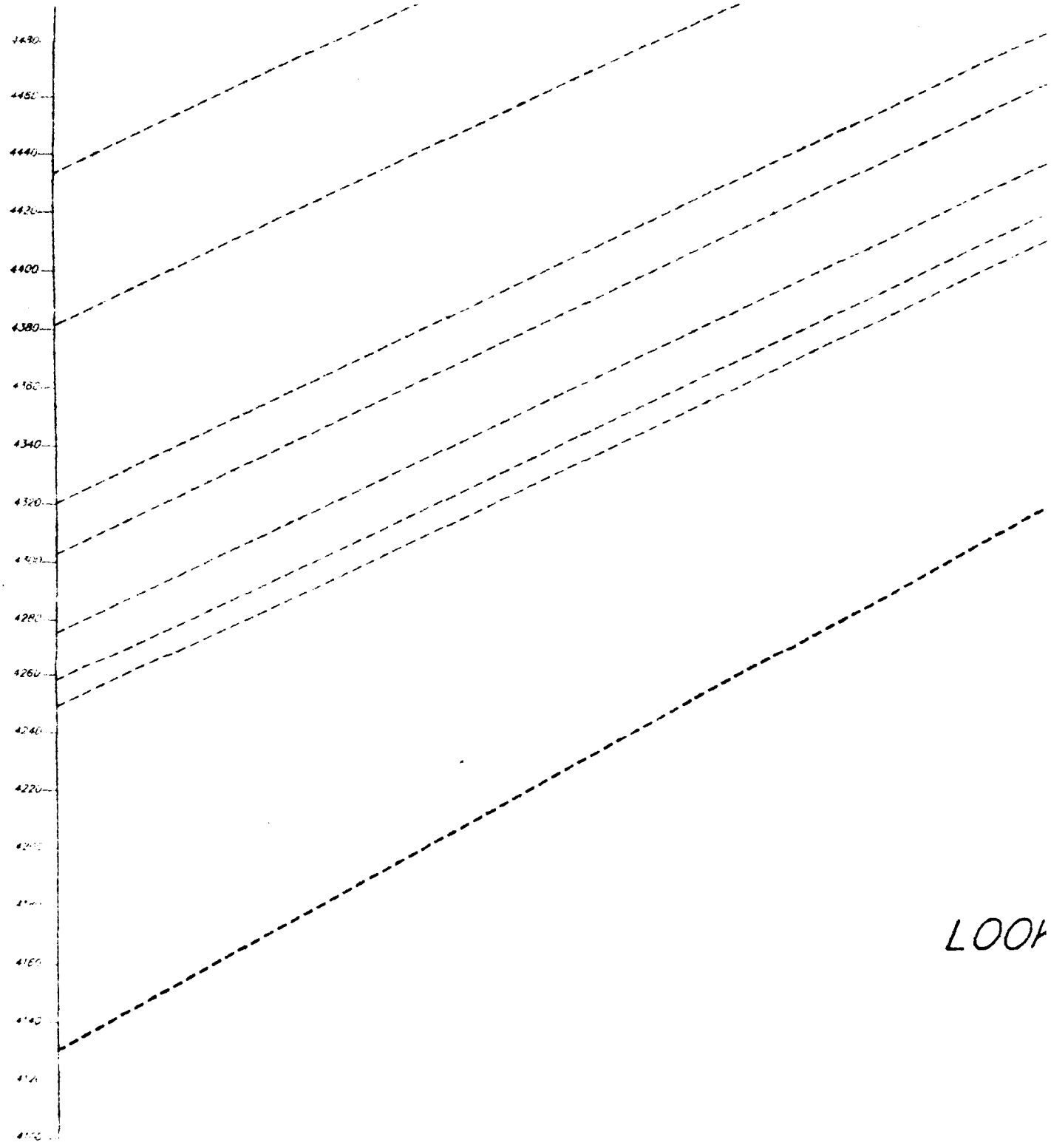

HIGH LEVEL
OUTLET WORKS
LIMESTONE
SILTSTONE
LIMESTONE
& SHALE
SHALE
LOW LEVEL
OUTLET WORKS
SPILLWAY
Shear Zone & Underlying
Altered & Solutioned Limestone
LIMESTONE
LIMESTONE & SHALE
4300
4480
4460
4440


ON-ON-3
LIMESTONE
LIMESTONE & SHALE
ARGILLACEOUS
ON-ON-2
SPILLWAY
℄


LIMESTONE &
SILTSTONE
COLLAPSE &
SHEAR BRECCI
CH-CH-24
CH-CH-4
SHALE
ARGILLACEOUS LIMESTONE
SHALE & LIMESTONE
LIMESTONE & SILTSTONE
LIMESTONE
SHALE & LIMESTONE
LIMESTONE
LIMESTONE
LIMESTONE & SHALE
SHALE & LIMESTONE


COLLAPSE &
EAR BRECCIA
CH-24
ON-ON-4
ON-ON-41
ON-ON-45
AUXILIARY SPILLWAY OGEE CREST
PALOMAS GRAVEL
SANTA FE GROUP
MADERA LIMESTONE


CN-QN-43
RAVEL
ONE
4760
4740
4720
4700
4680
4660
4640
4620
4600
4580
4560
4540
4520
4500
4480
4460
4440

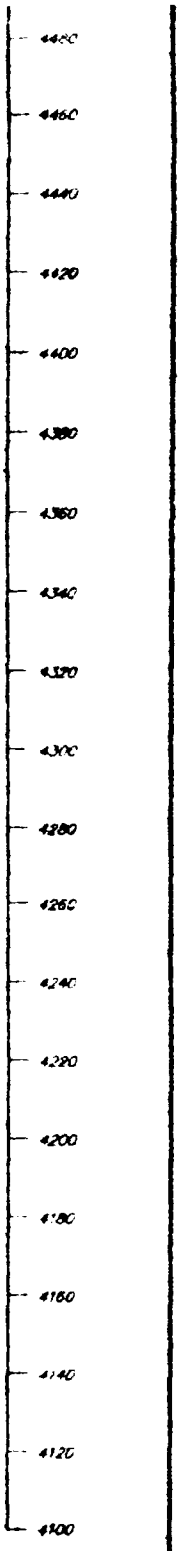

4480
4460
4440
4420
4400
4380
4360
4340
4320
4300
4280
4260
4240
4220
4200
4180
4160
4140
4120
4100
LOOK



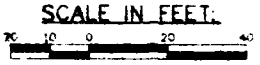
LOOKING S 27° E


BEND IN SECTION

LOOKING S 14° E


4480
4460
4440
4420
4400
4380
4360
4340
4320
4300
4280
4260
4240
4220
4200
4180
4160
4140
4120
4100

SCALE IN FEET:



LEGEND:



CONTACT BETWEEN MAPPED ROCK UNITS, DASHED WHERE ASSUMED

FAULTS SHOWING REATIVE MOVEMENTS, DASHED WHERE ASSUMED

FINAL EXCAVATED SURFACE

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

DESIGNED BY: C.B.D.

DRAWN BY: S.D.M.

CHECKED BY: W.S.B.

RIO GRANDE & TRIBUTARIES — TRUTH OR CONSEQUENCES, N.M.

CUCHILLO NEGRO DAM

FOUNDATION REPORT

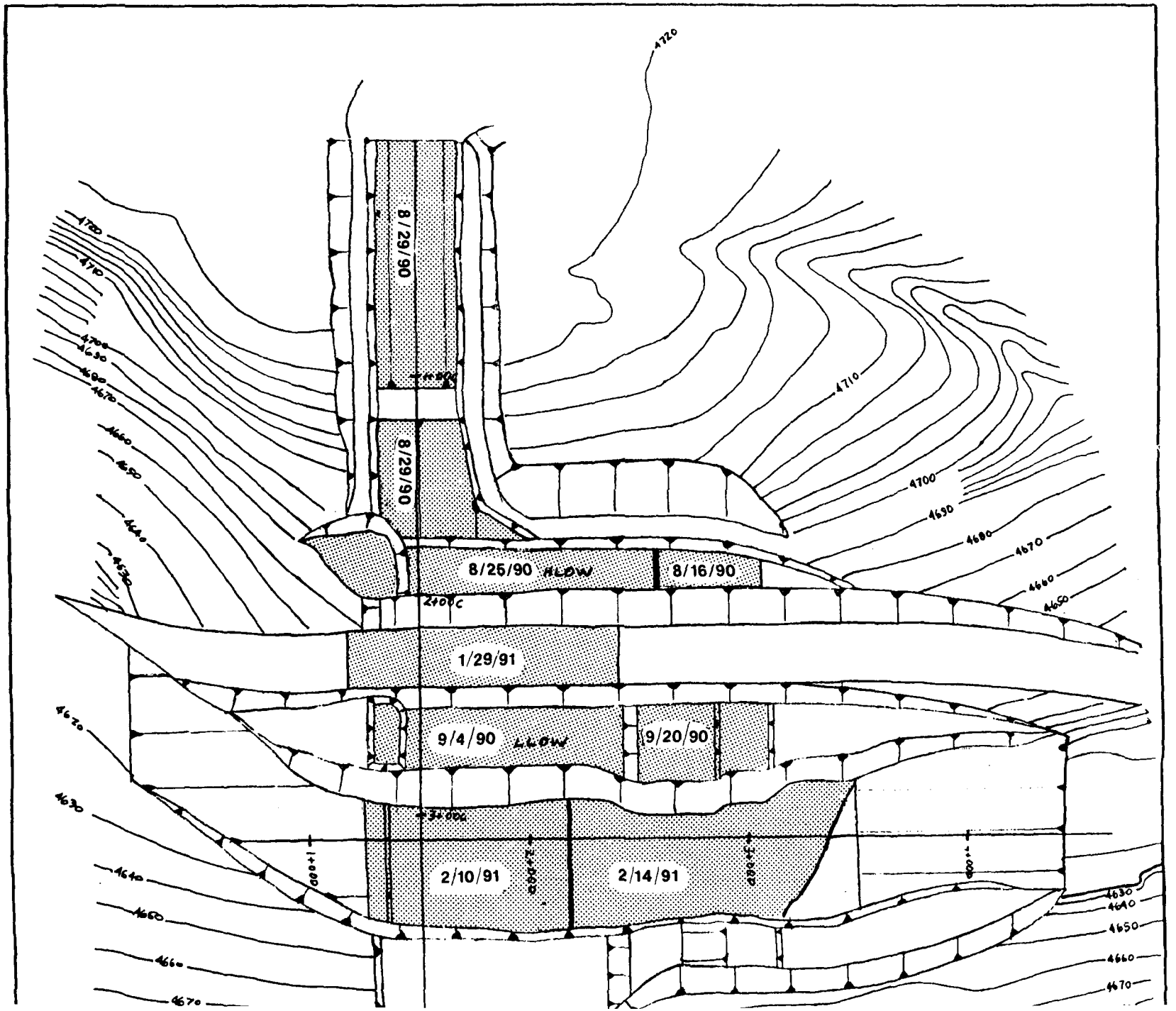
GEOLOGIC CROSS SECTION THROUGH RCC DAM AND AUXILIARY SPILLWAY

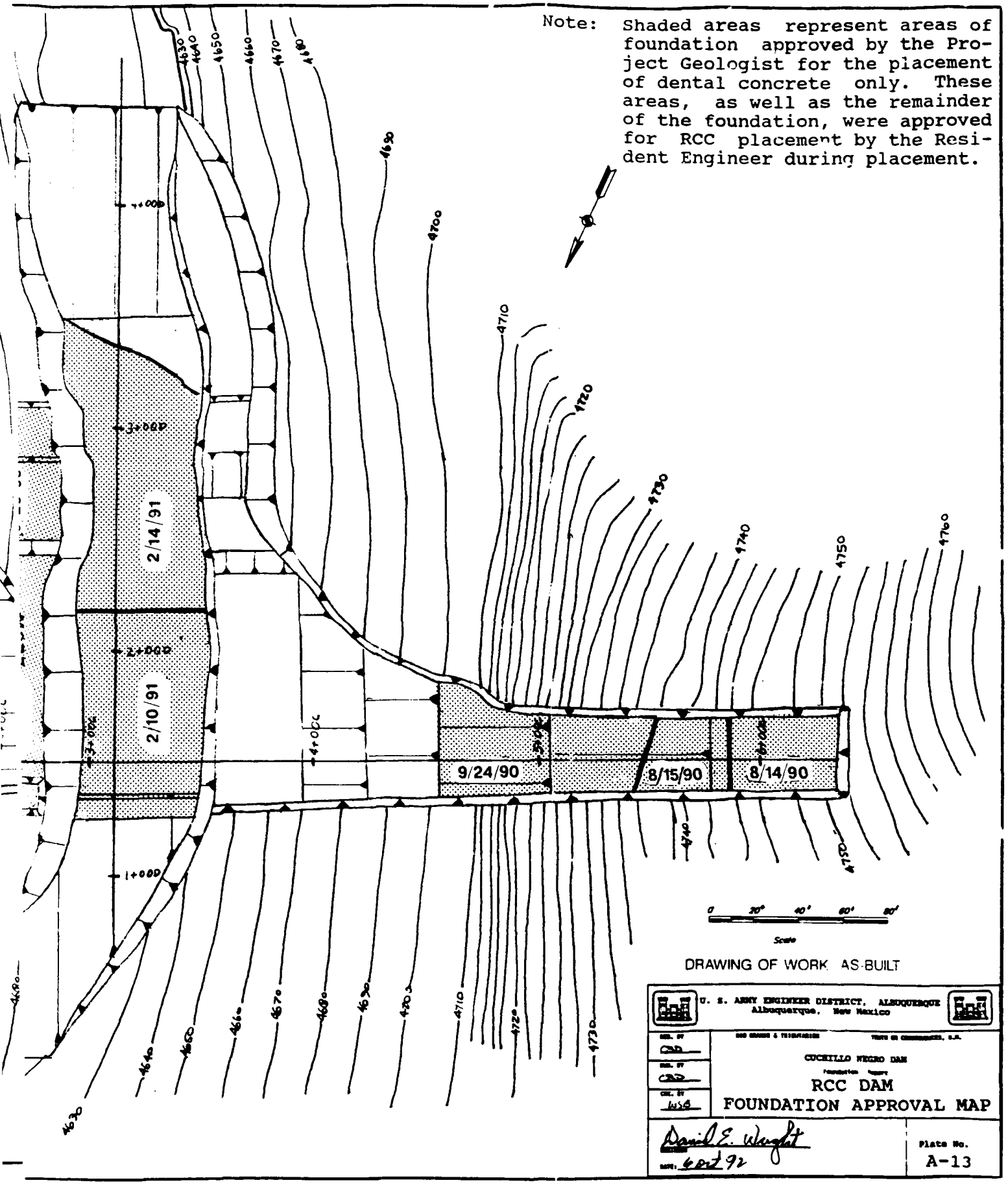
David E. Wright
ENGINEER

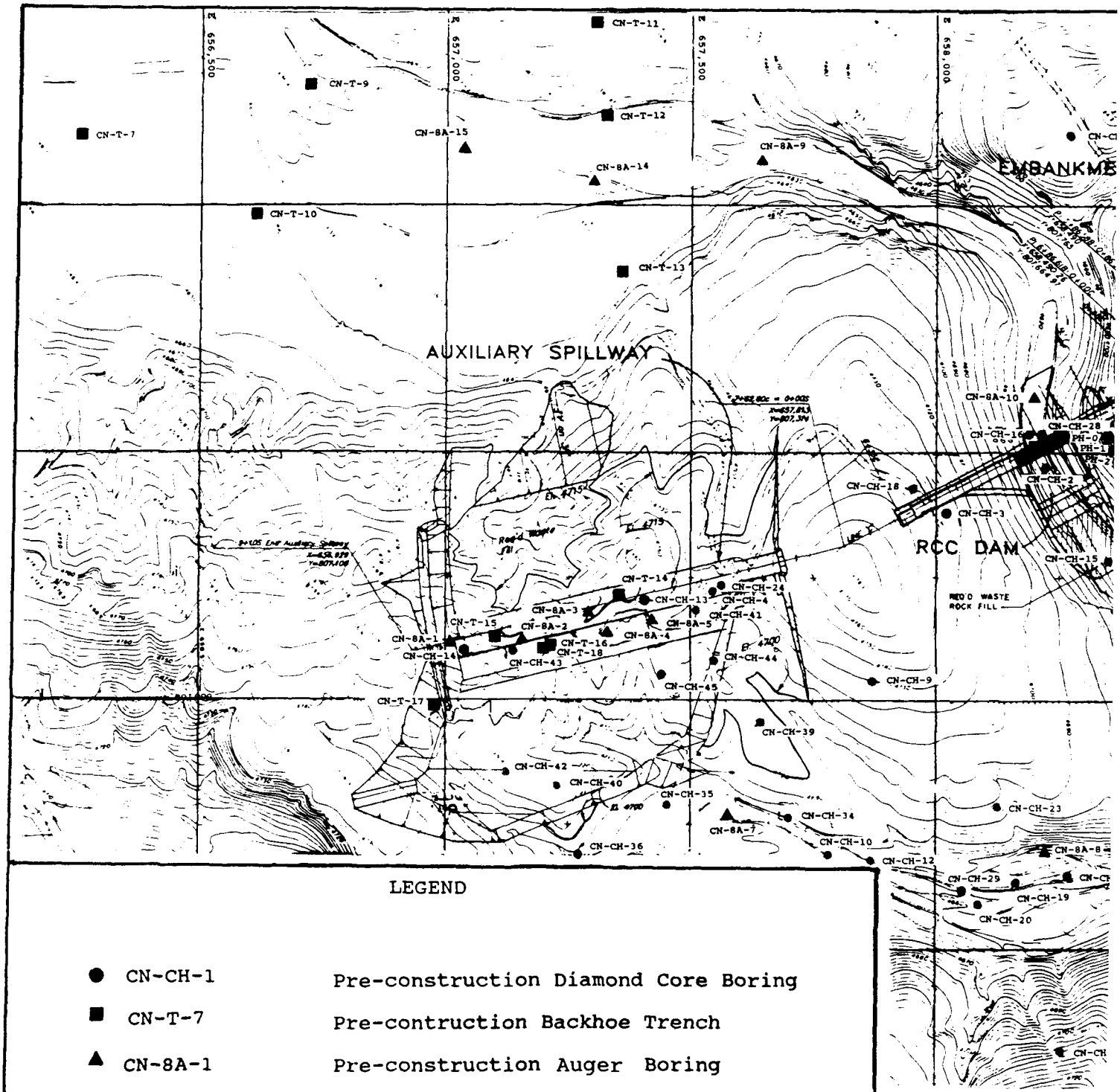
DATE:

FILE NUMBER

PLATE A-12

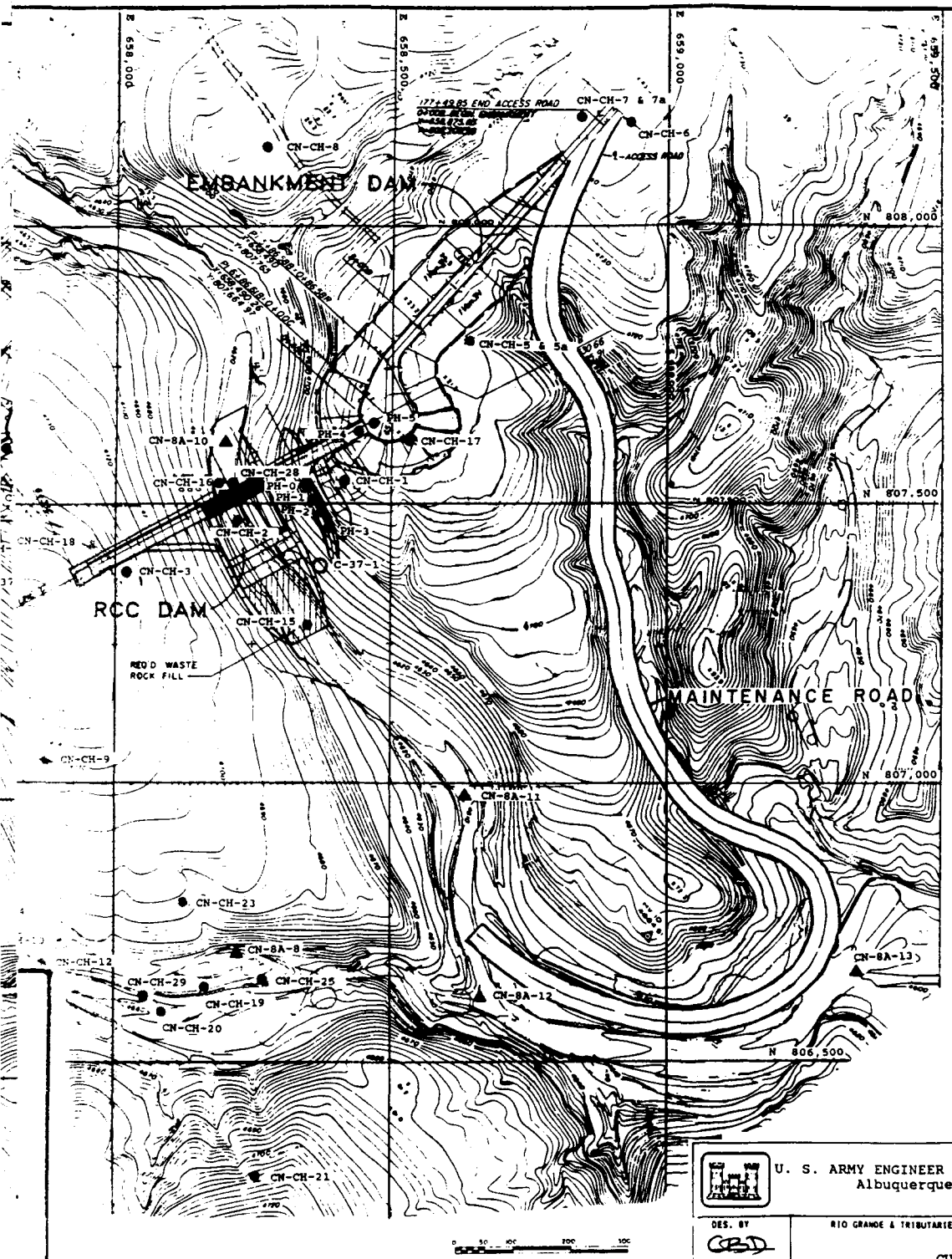

8/29/90
8/29/90
8/25/90 FLOW
8/16/90
1/29/91
9/4/90 FLOW
9/20/90
2/10/91
2/14/91
4720
4710
4700
4690
4680
4670
4660
4650
4640
4630
4620


Note: Shaded areas represent areas of foundation approved by the Project Geologist for the placement of dental concrete only. These areas, as well as the remainder of the foundation, were approved for RCC placement by the Resident Engineer during placement.
2/14/91
2/10/91
9/24/90
8/15/90
8/14/90
1+000
2+000
3+000
4+000
0 20' 40' 60' 80'
Scale
DRAWING OF WORK AS-BUILT
U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
Albuquerque, New Mexico
CUCHILLO NEGRO DAM
RCC DAM
FOUNDATION APPROVAL MAP
David E. Wright
Apr 92
Plate No.
A-13



LEGEND

● CN-CH-1	Pre-construction Diamond Core Boring
■ CN-T-7	Pre-contruction Backhoe Trench
▲ CN-8A-1	Pre-construction Auger Boring
⬢ CN-CH-13	Pre-construction Rotary Boring
C-37-1	Construction Denison Boring
● PH-1	Construction Air Percussion Boring

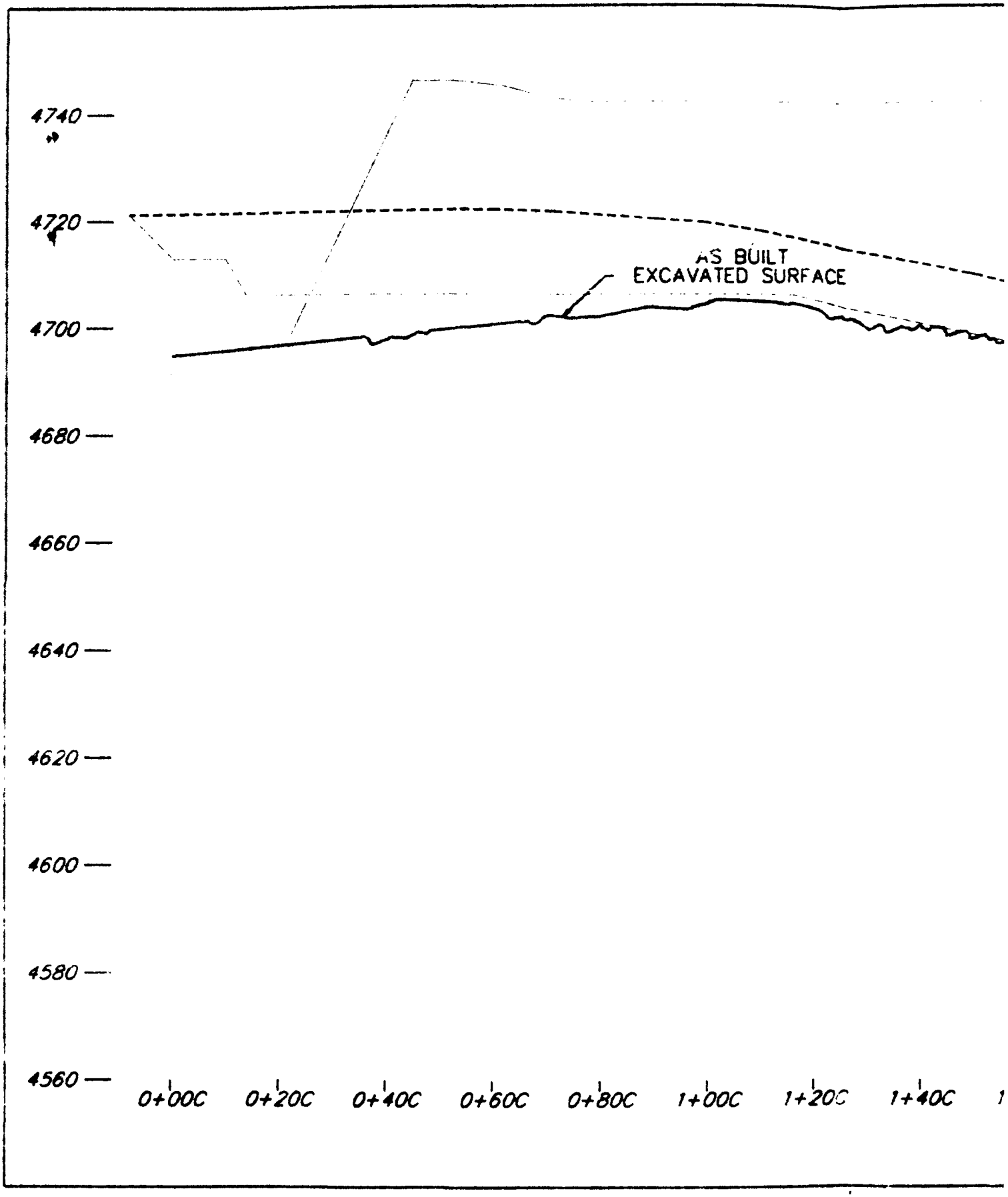


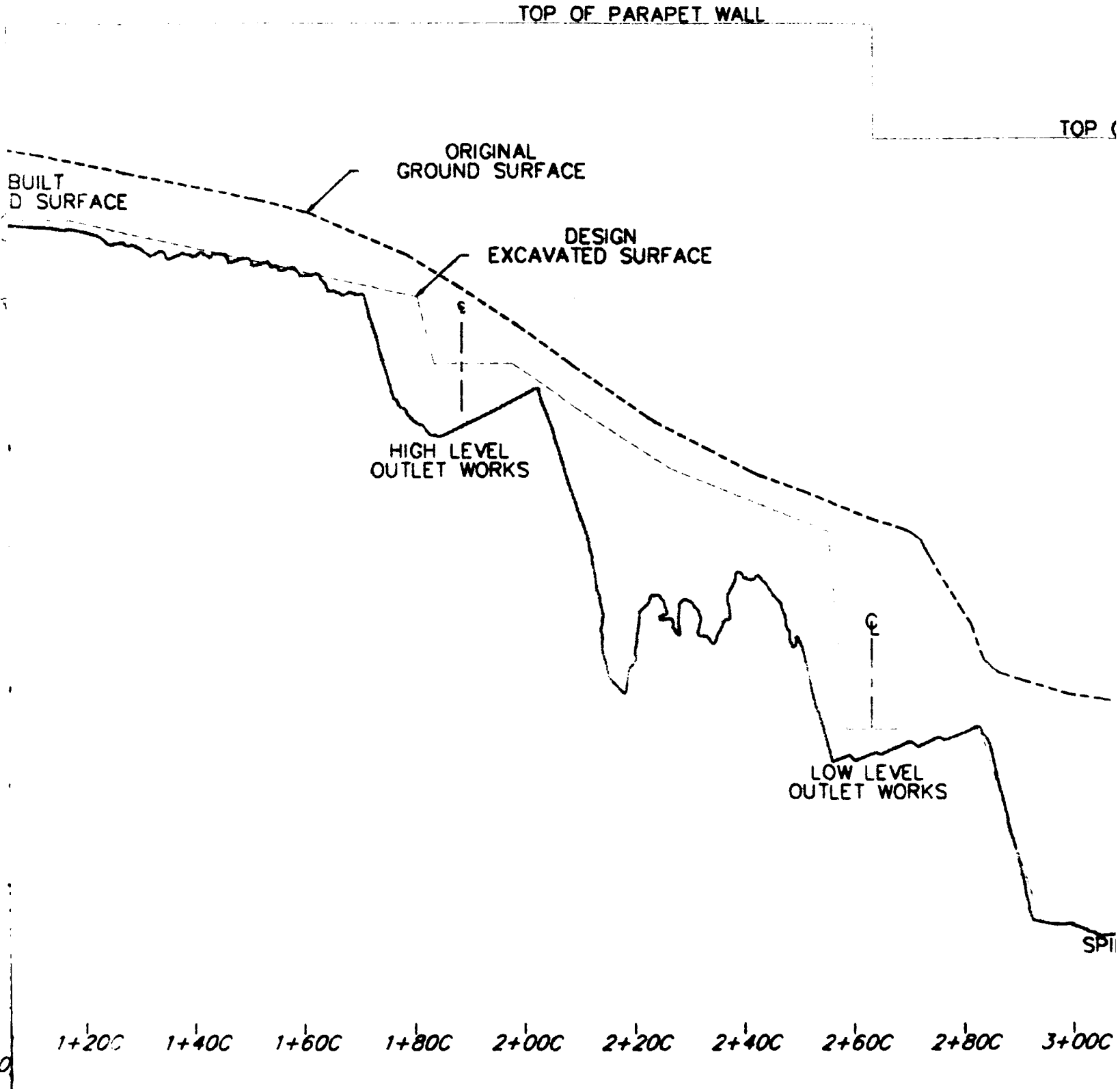
U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
Albuquerque, New Mexico

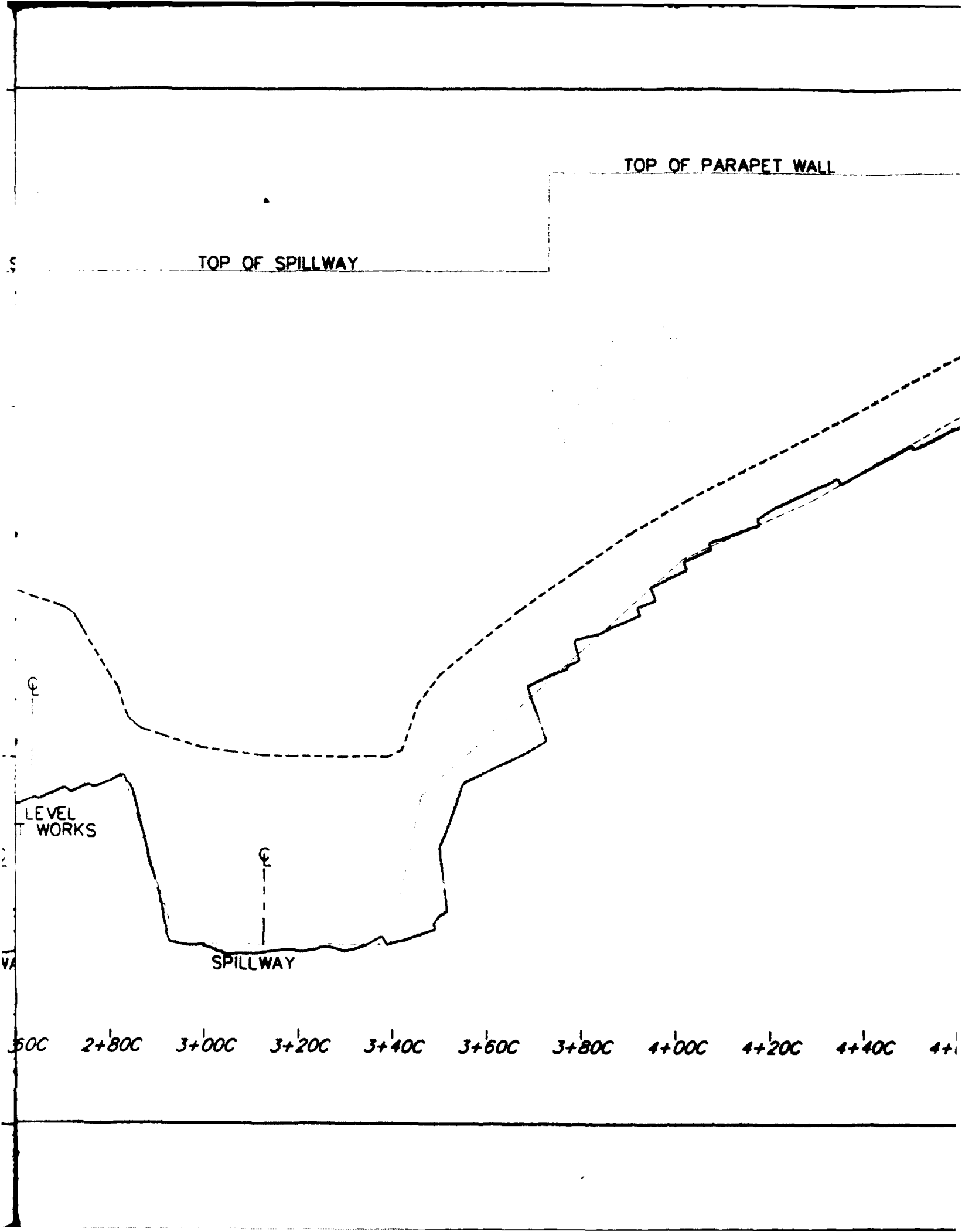
DES. BY CBD	RIO GRANDE & TRIBUTARIES — TRUTH OR CONSEQUENCES, N.M.
DWN. BY CBD	CUCHILLO NEGRO DAM
CHK. BY DEL	Foundation Report
	PLAN OF EXPLORATIONS

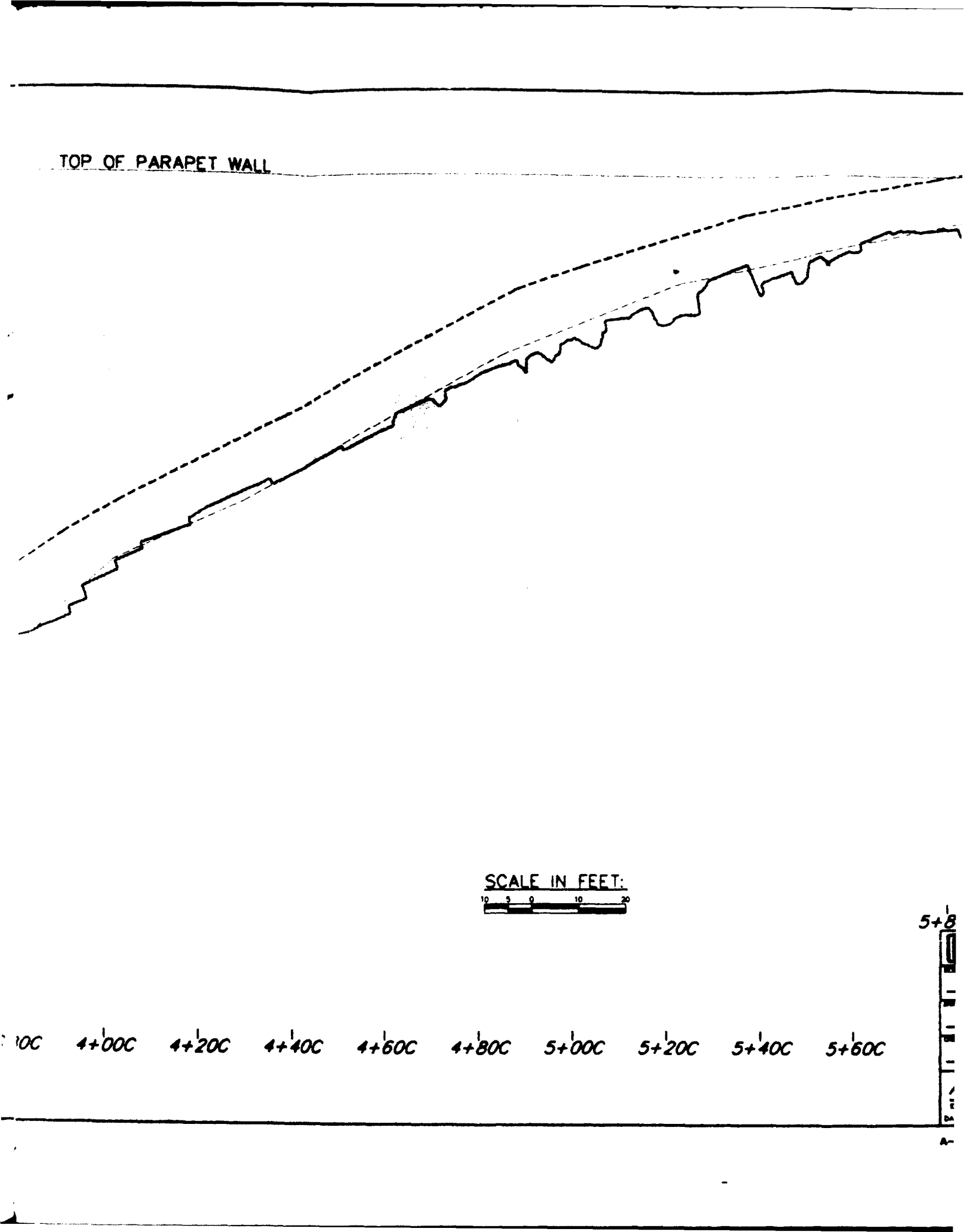
David E. Wright
ENGINEER
DATE: 9-4-92

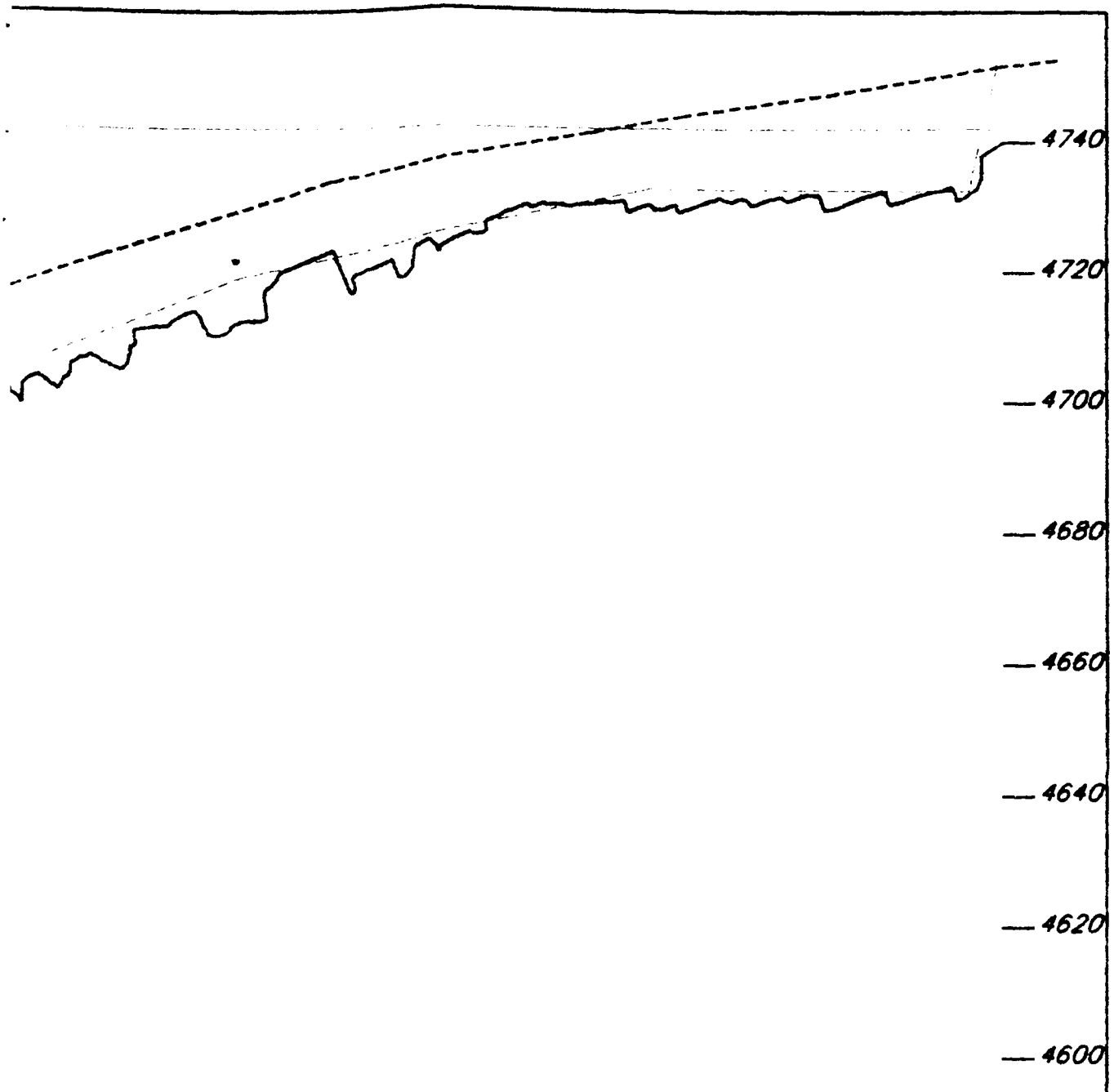
Plate No.
A-14


4740
4720
4700
4680
4660
4640
4620
4600
4580
4560
AS BUILT
EXCAVATED SURFACE
0+00C
0+20C
0+40C
0+60C
0+80C
1+00C
1+20C
1+40C


TOP OF PARAPET WALL
TOP
ORIGINAL
GROUND SURFACE
BUILT
D SURFACE
DESIGN
EXCAVATED SURFACE
HIGH LEVEL
OUTLET WORKS
LOW LEVEL
OUTLET WORKS
SPI
1+20C
1+40C
1+60C
1+80C
2+00C
2+20C
2+40C
2+60C
2+80C
3+00C


TOP OF PARAPET WALL
TOP OF SPILLWAY
℄
LEVEL
T WORKS
℄
SPILLWAY
2+80C
3+00C
3+20C
3+40C
3+60C
3+80C
4+00C
4+20C
4+40C


TOP OF PARAPET WALL
SCALE IN FEET:
10 5 0 10 20
5+8
4+00C
4+20C
4+40C
4+60C
4+80C
5+00C
5+20C
5+40C
5+60C



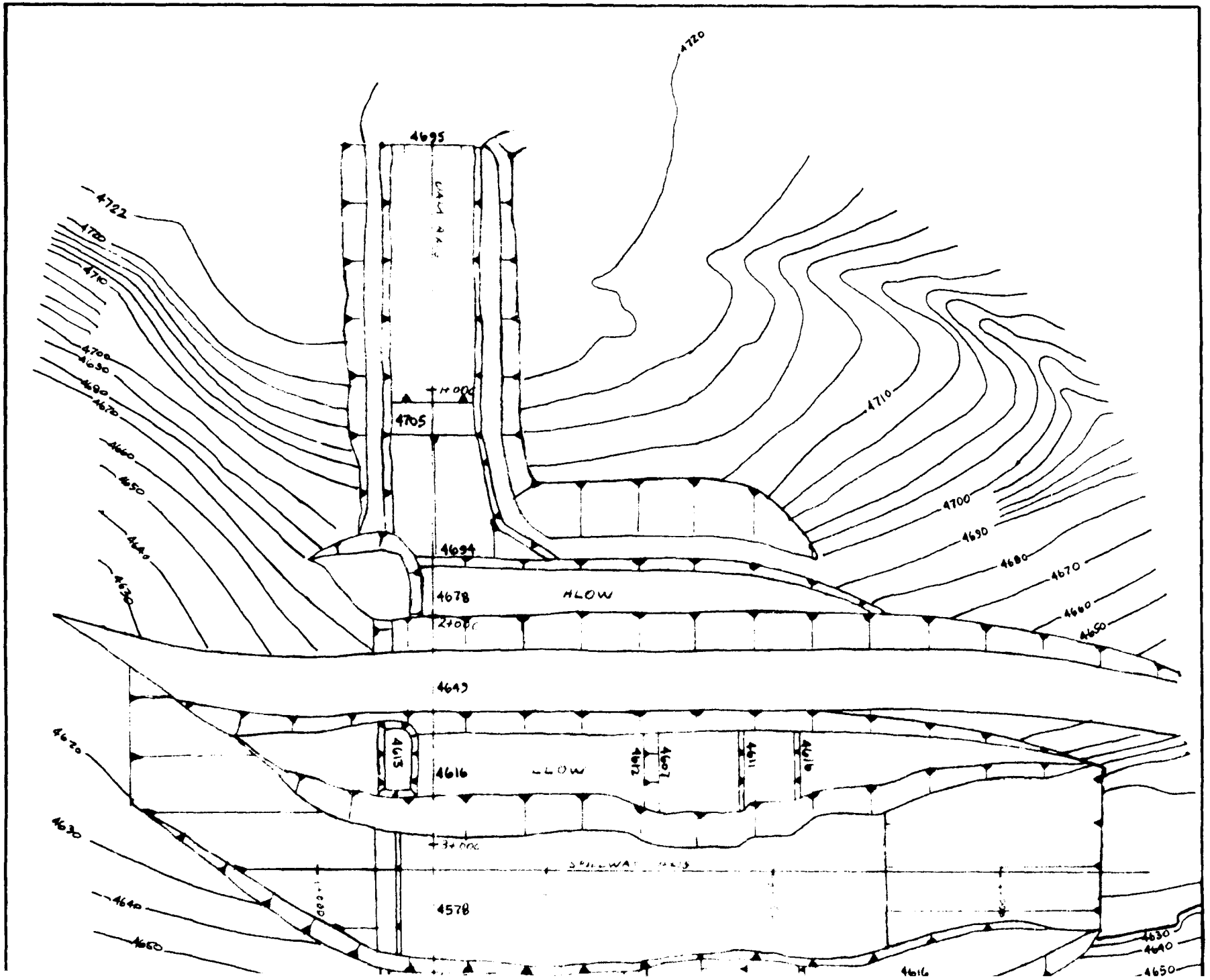
E IN FEET:
0 10 20

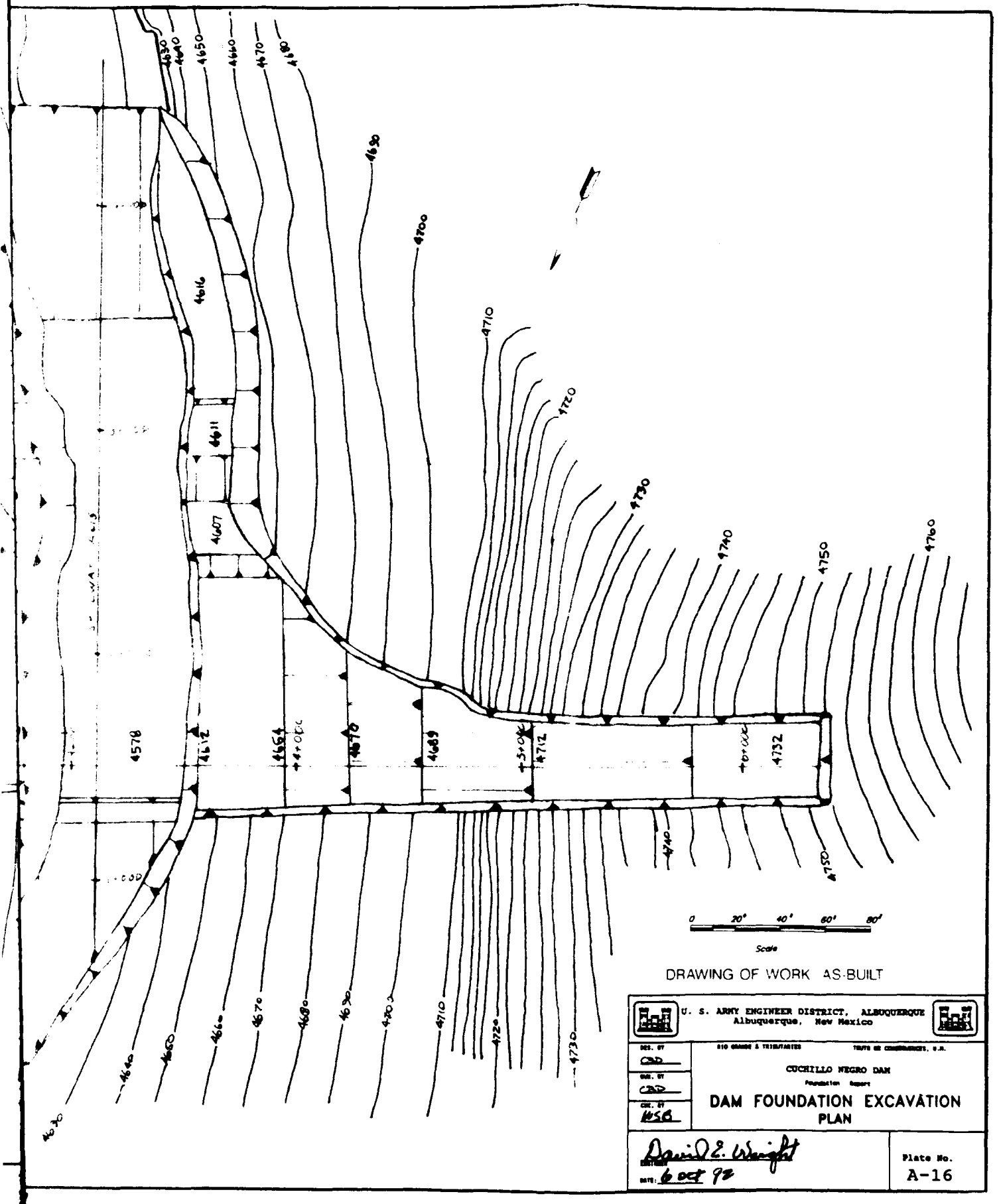
U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

CUCHILLO NEGRO DAM
FOUNDATION REPORT

RCC DAM
DESIGN VS. AS BUILT
EXCAVATION SURFACES

DESIGNED BY: CBA
DRAWN BY: GAH
CHECKED BY: GAH
ENGINEER: David E. Wright
DATE: 10-5-92
FILE NUMBER:
PLATE A-16


4720
4695
4722
4720
4710
4700
4690
4680
4670
4660
4650
4640
4630
4705
4694
4678
4643
4613
4616
4607
4611
4618
4578
4630
4640
4650



DRAWING OF WORK AS-BUILT

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
Albuquerque, New Mexico

DES. BY: CAD
DRN. BY: CAD
CHK. BY: MSG

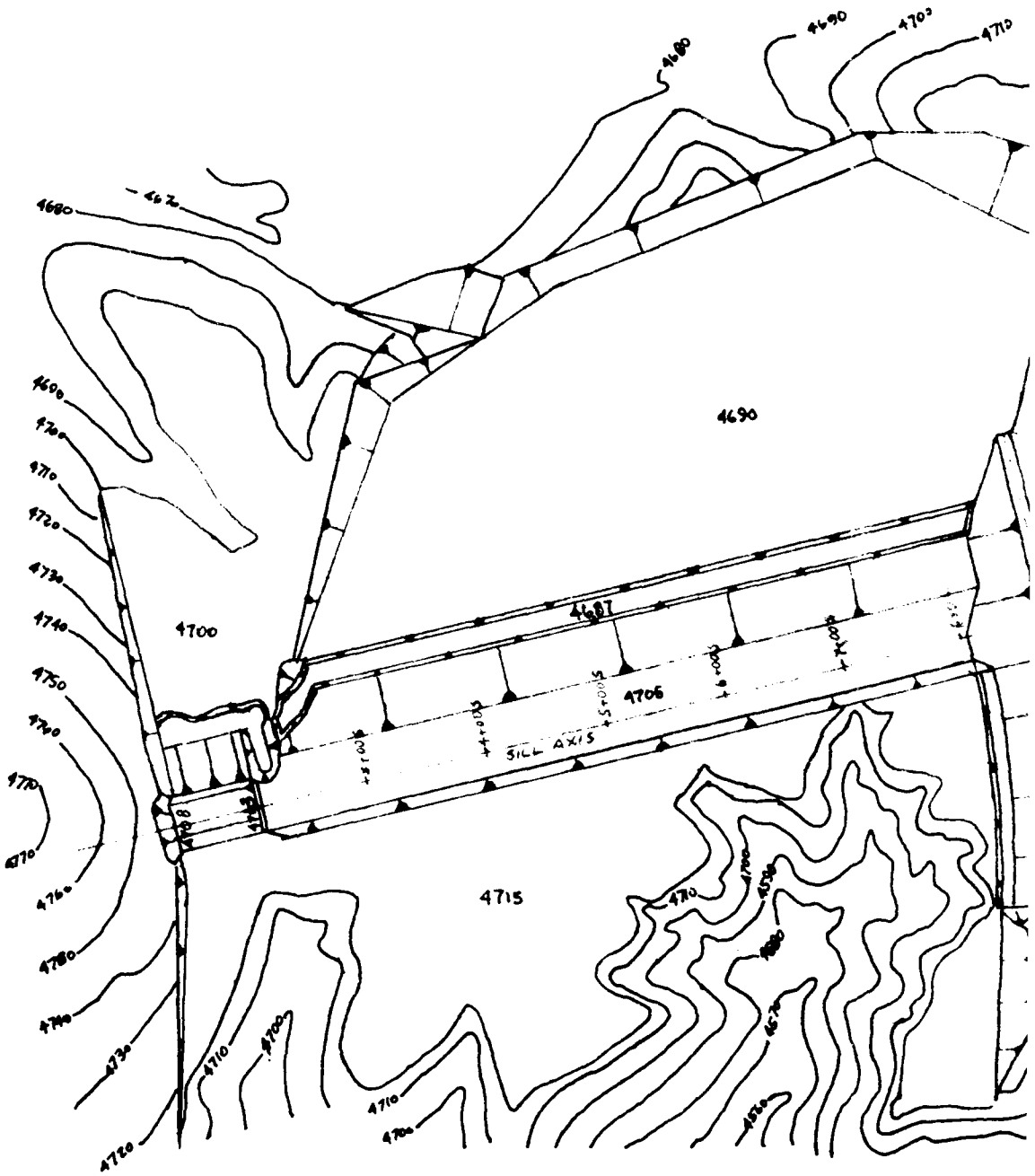
RIO GRANDE & TRIBUTARIES — TEXAS AND CHIHUAHUA, U.S.

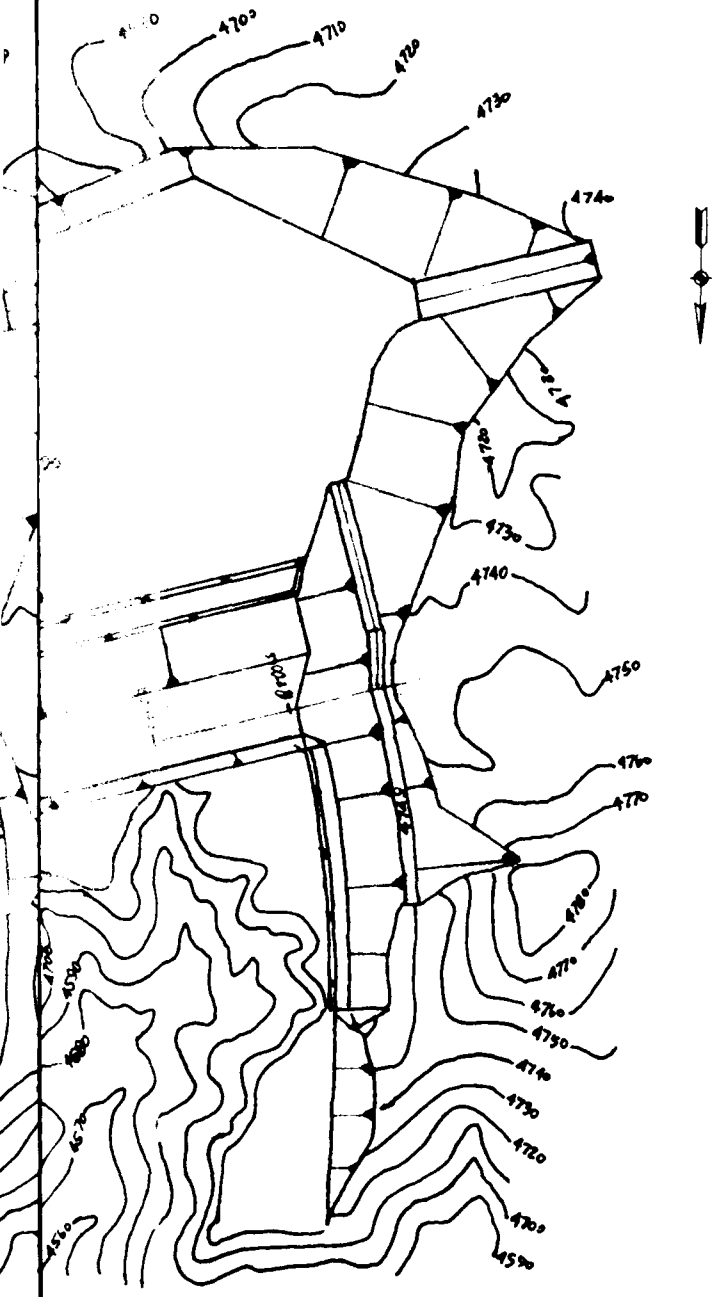
CUCHILLO NEGRO DAM
Foundation Report

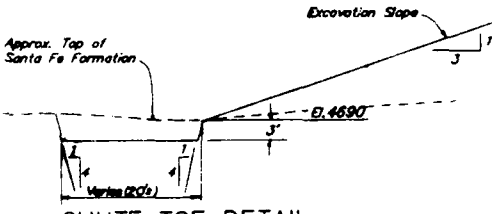
DAM FOUNDATION EXCAVATION PLAN

David E. Wright
DATE: 6 Oct 92

Plate No.
A-16


4680
4690
4700
4710
4690
4700
4706
4715
SILL AXIS
4720
4730
4740
4750
4760
4770
4780
4790
4710
4700
4710
4700





CHUTE TOE DETAIL

Scale: 1"=25'

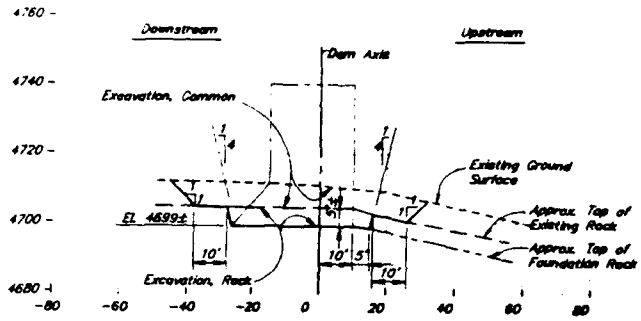
Scale: 0 50' 100' 150' 200'

DRAWING OF WORK AS BUILT

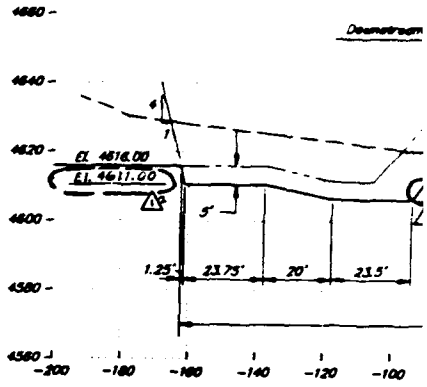
U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
Albuquerque, New Mexico

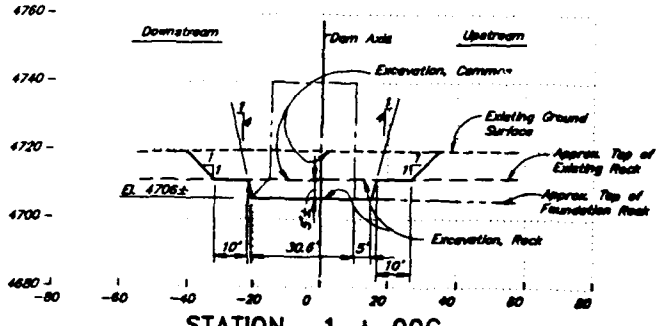
DES. BY CAD	RIO GRANDE & TRIBUTARIES — TRUTH OR CONSEQUENCES, N.M.
DRN. BY CAD	CUCHILLO NEGRO DAM
CHK. BY WSG	Foundation Report AUXILIARY SPILLWAY FOUNDATION EXCAVATION PLAN
CONTRACT DATE: Sept 92	Plate No. A-17

David E. Wright

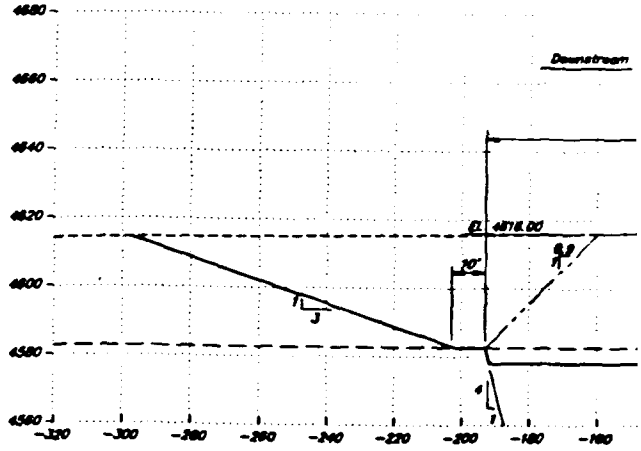


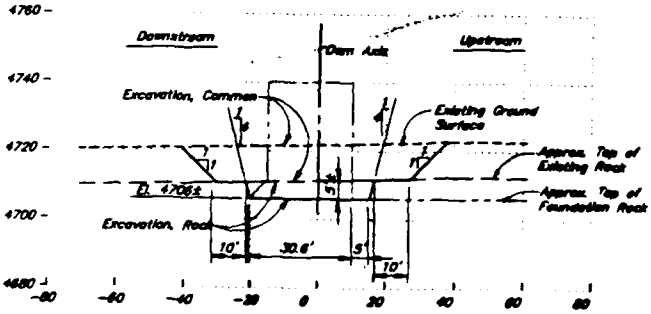
STATION 1 + 50C



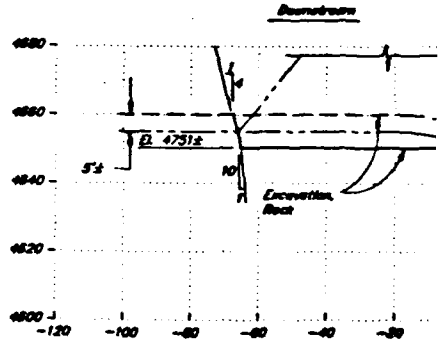


STATION 1 + 00C

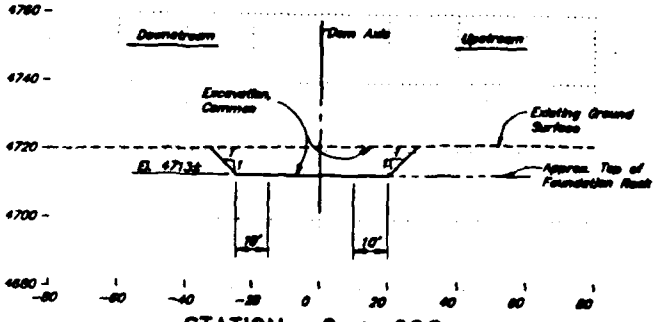




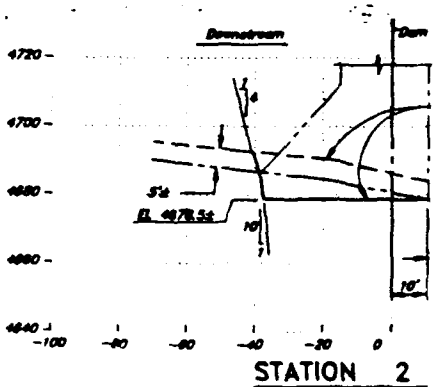
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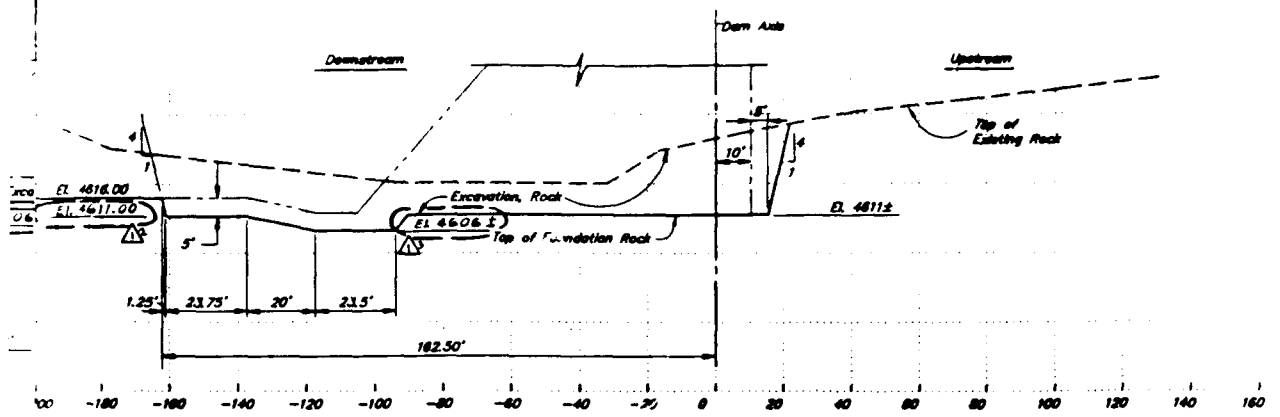
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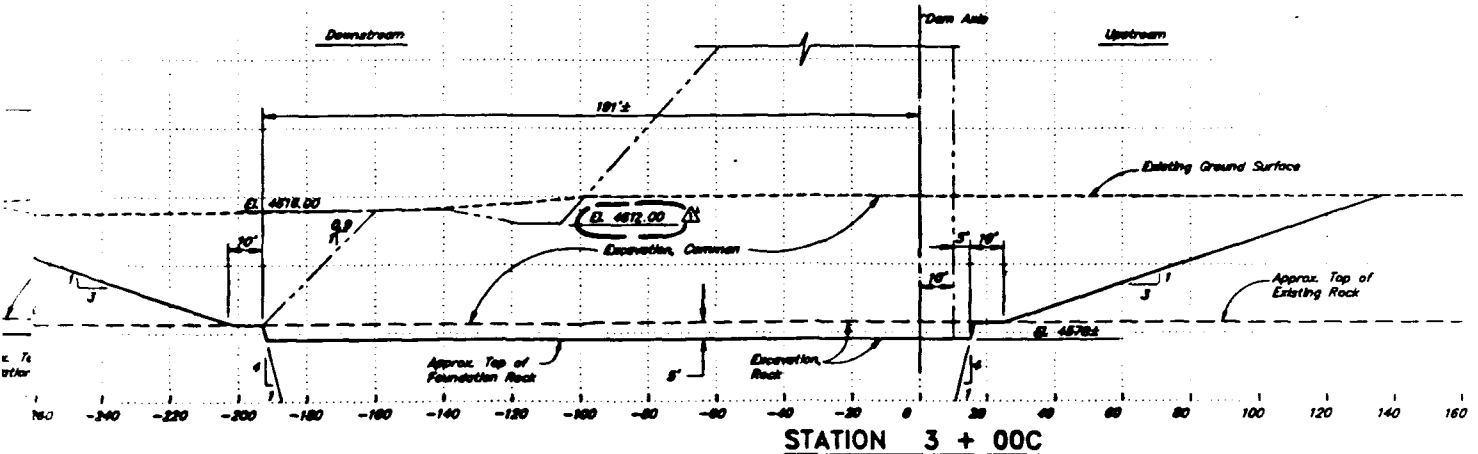
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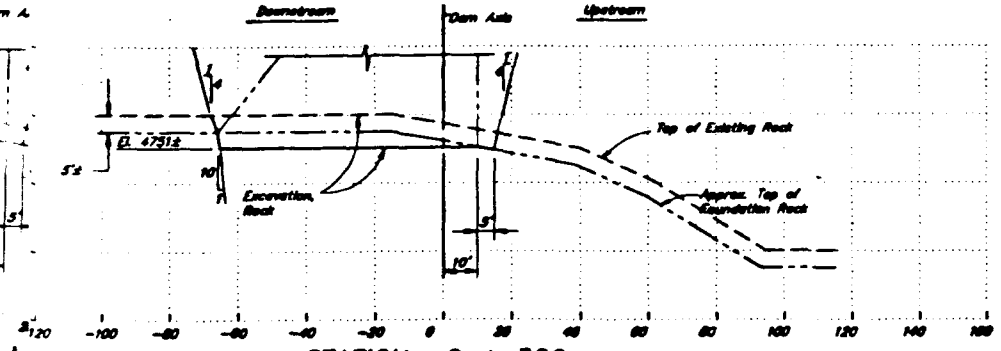
STATION 2



STATION 3 + 50C



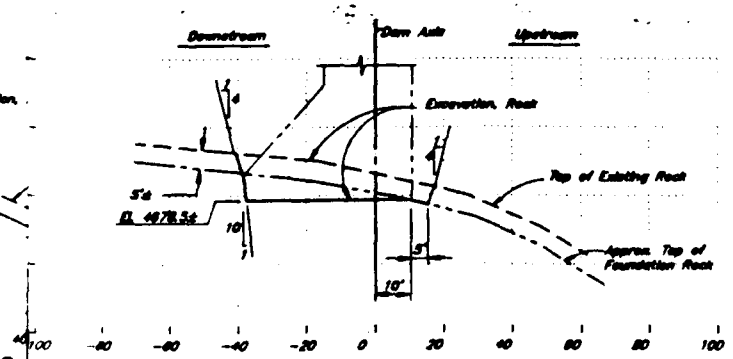
STATION 3 + 00C



STATION 2 + 50C

NOTE: ⚠4
All Rock Slopes Steeper Than 1V To 1H Formed By Blasting Shall Be Pre Split.

0 20 40 60 80
Scale 1" = 20'



STATION 2 + 00C

	AND ADDED NOTE (NO. 2)	AUG 89	
⚠4	REVISED ELEVATION, ADDED ELEVATIONS		
SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

BOYLE ENGINEERING CORPORATION
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

DESIGNED BY TBP
DRAWN BY SBV
CHECKED BY RMJ

RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

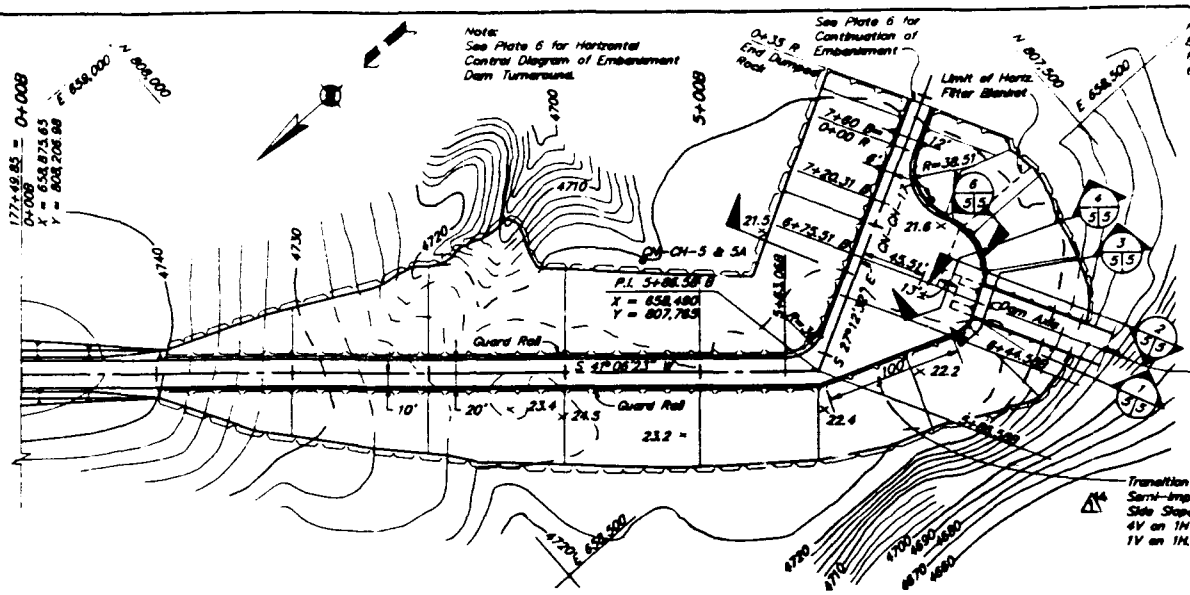
RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

DAM FOUNDATION EXCAVATION SECTIONS

Fred Burns
ENGINEER
DATE JULY, 1989

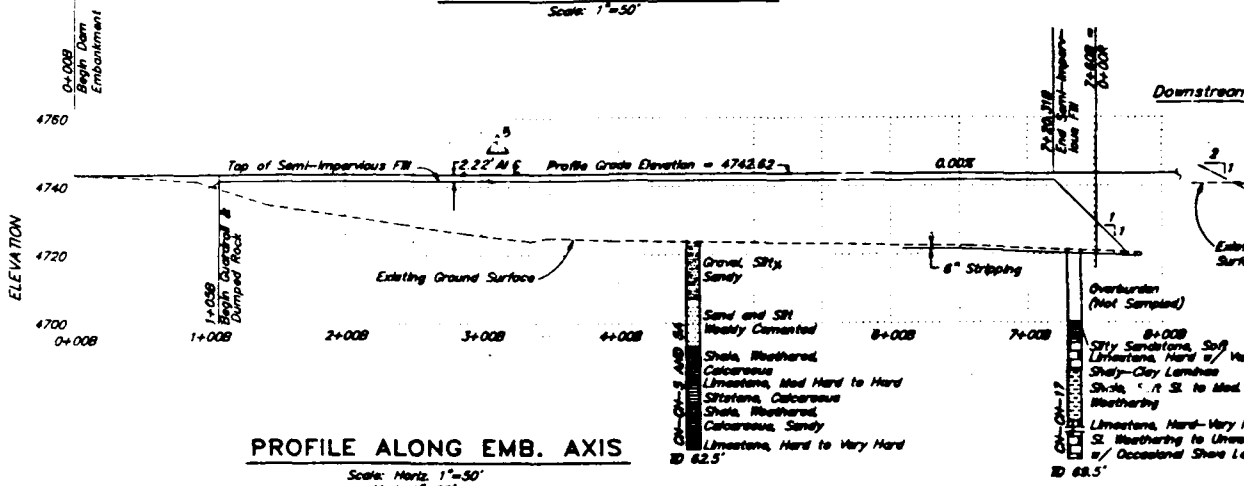
SOL.NO.DACW47-89-B-0024
FILE NUMBER RG-TC-A-9.1

PLATE A-18



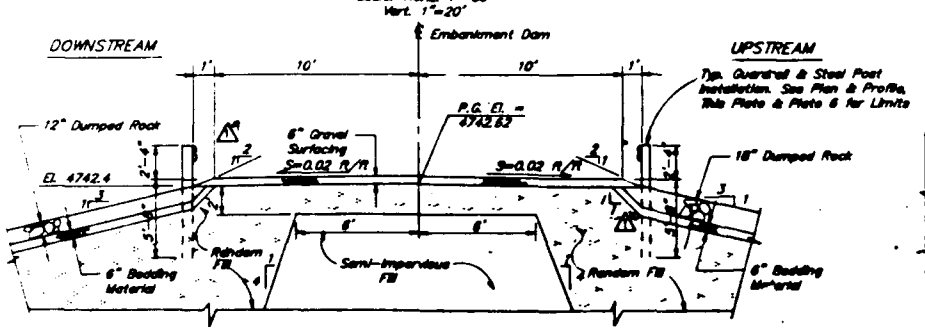
EMBANKMENT DAM PLAN

Scale: 1"=50'



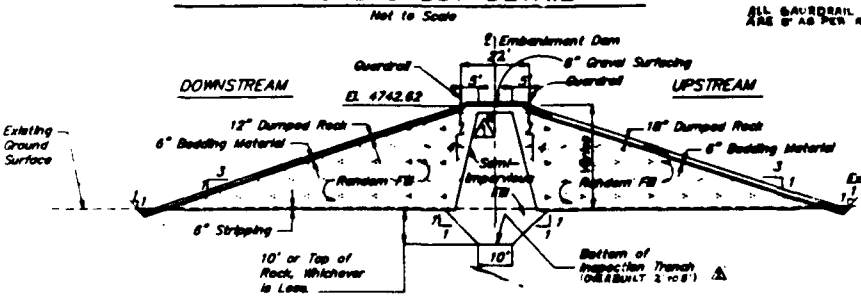
PROFILE ALONG EMB. AXIS

Scale: Horiz. 1"=50'
Vert. 1"=20'



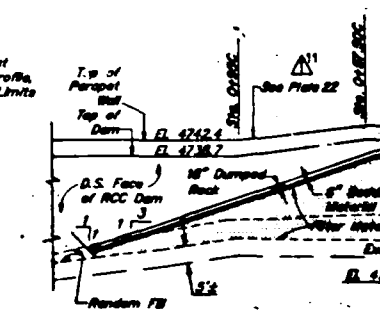
STA. 1+05B to STA. 5+63.06B TYPICAL CREST DETAIL

Not to Scale



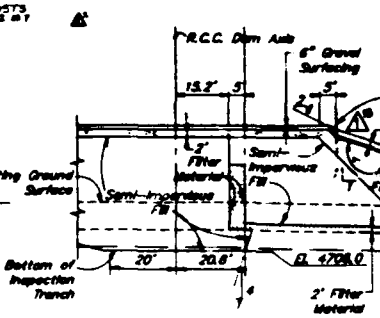
STA. 0+00B to STA. 5+63.06B TYPICAL EMBANKMENT DAM SECTION

Scale: 1"= 20'



SECTION

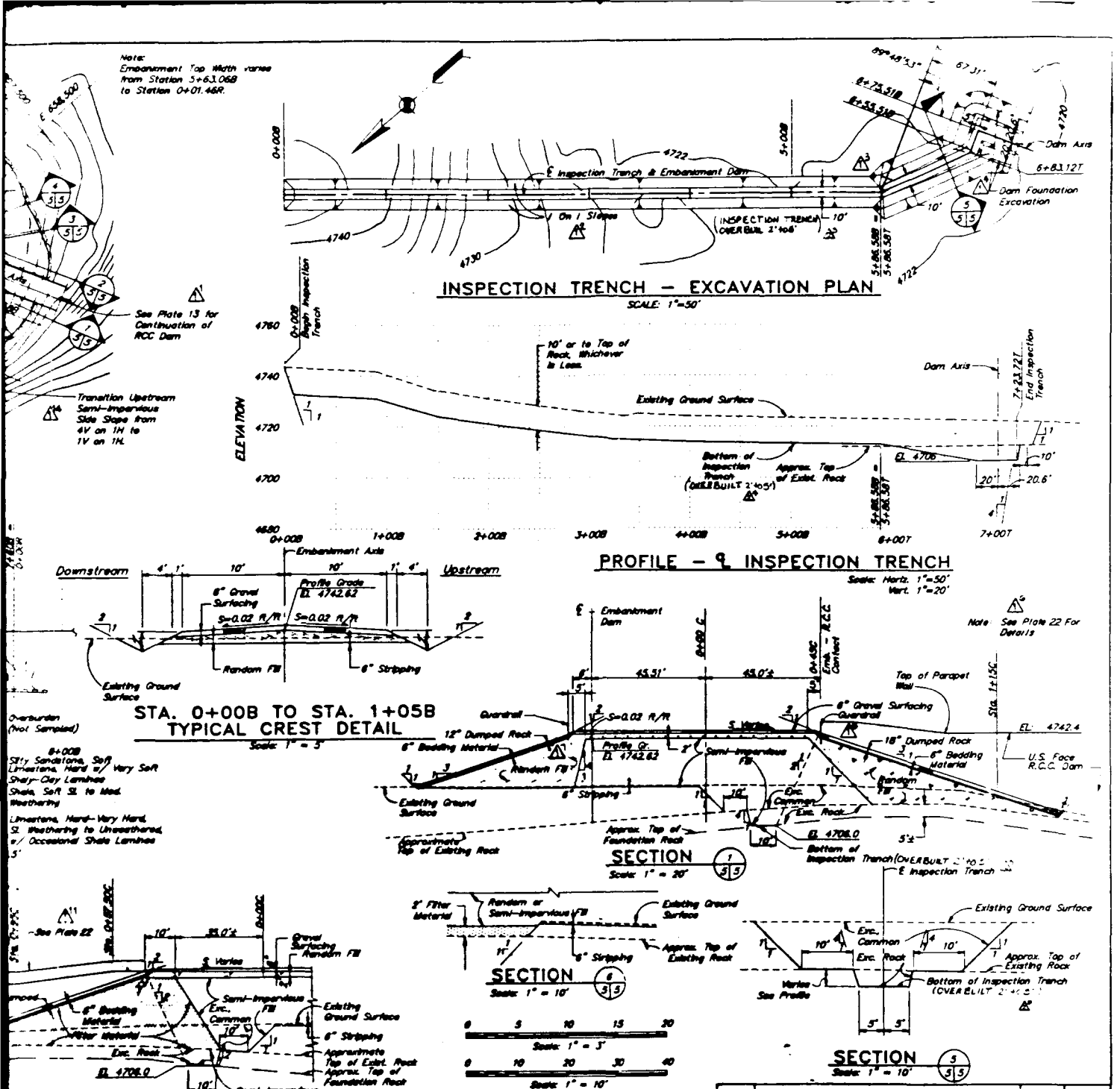
Scale: 1" = 20'



SECTION

Scale: 1" = 20'

VALUE ENGINEERING PR



SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	RAIL POSTS AS PER RFI #7 "AS BUILT"	7/29/91	
	REVISED INSPECTION TRENCHES & GUARD		
	ADDED NOTES, DIMENSION & REVISED SLOPE OF RIPRAP ROCK AT SHOULDER. (AM #2)	AUG '89	

REVISIONS

BOYLE ENGINEERING CORPORATION
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

EMBANKMENT DAM
PLAN, PROFILE SECTIONS AND DETAILS

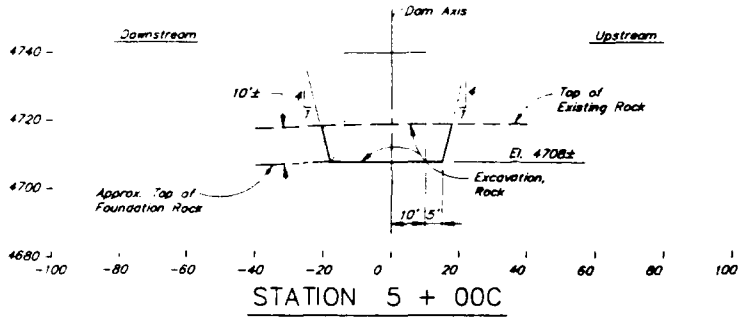
DESIGNED BY: RMJ
DRAWN BY: GEA
CHECKED BY: TMP

ENGINEER
DATE JULY, 1989

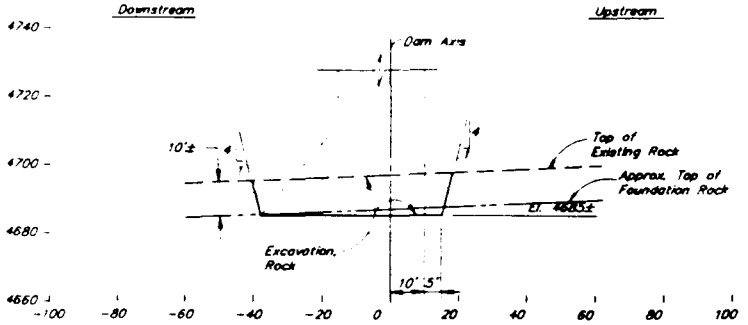
SOL. NO. DACW47-89-B-0024 PLATE
FILE NUMBER RG-TC-A-5 A-

VALUE ENGINEERING PROPOSALS MEAN HIGHER PROFITS

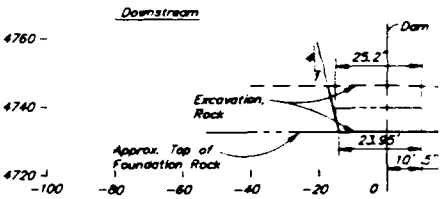
DRAWING OF WORK AS BUILT A-20



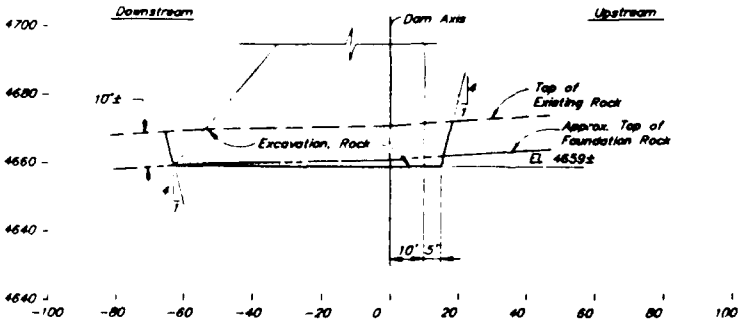
STATION 5 + 00C



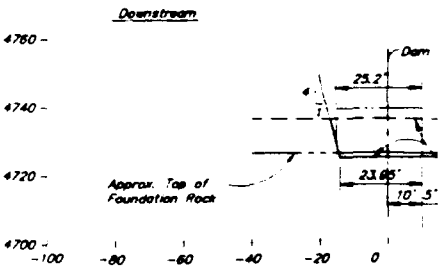
STATION 4 + 50C



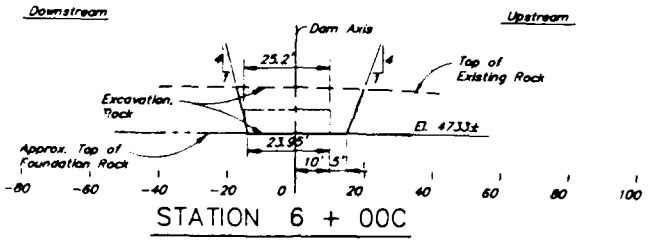
STATION 6 +



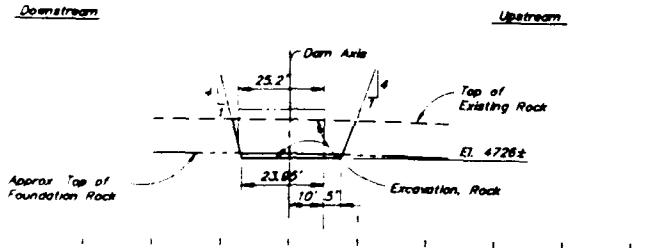
STATION 4 + 00C



STATION 5 +

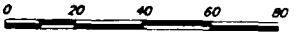


STATION 6 + 00C



STATION 5 + 50C

△ All Rock Slopes Steeper Than 1 / To 1 H Formed By Blasting Shall Be Pre Split



Scale: 1" = 20'

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
△	ADDED NOTE (AM # 2)	JUL 89	

REVISIONS

BOYLE ENGINEERING CORPORATION CONSULTING ENGINEERS ALBUQUERQUE, NEW MEXICO	U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE CORPS OF ENGINEERS ALBUQUERQUE, NEW MEXICO

DESIGNED BY: TWP
DRAWN BY: SBH
CHECKED BY: HMJ

RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

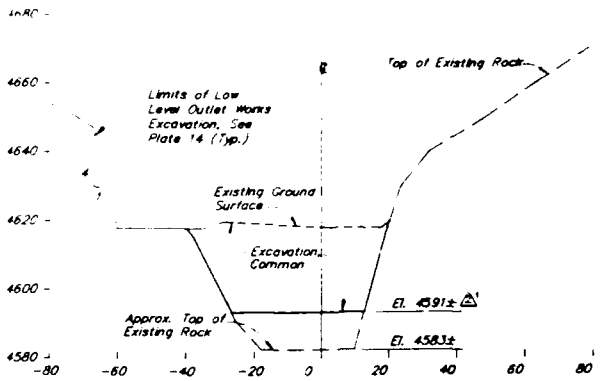
RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

DAM FOUNDATION EXCAVATION SECTIONS

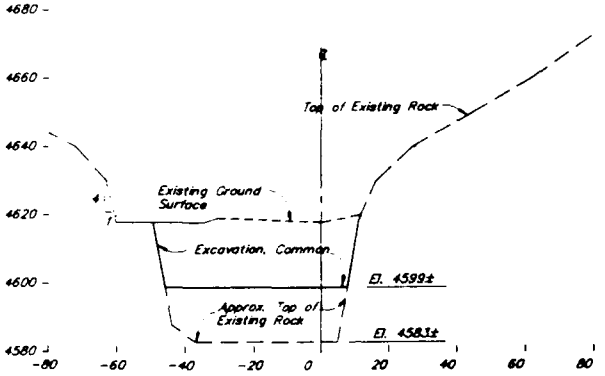
ENGINEER
DATE JULY 1989

SOL.NO.DACW47-89-[illegible] PLATE

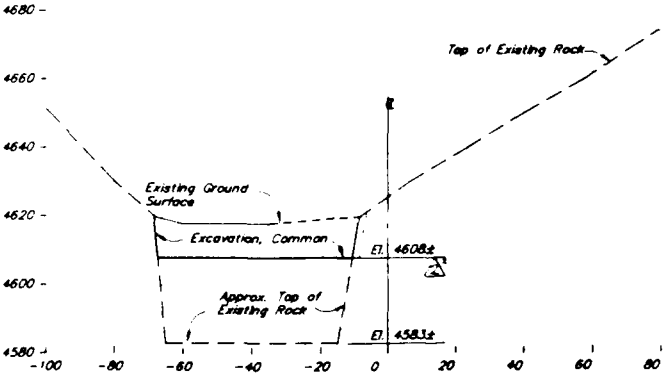
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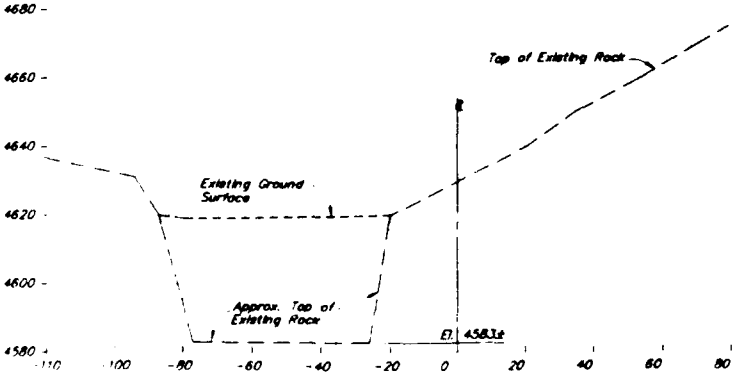
STATION 1 + 00D



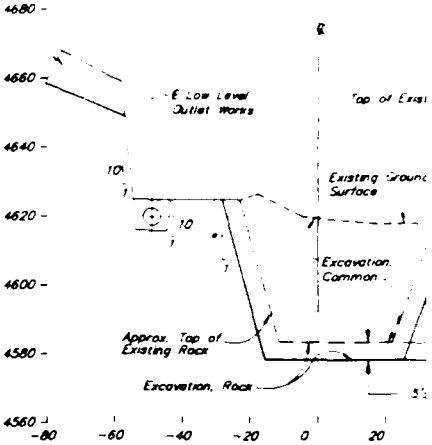
STATION 0 + 75D



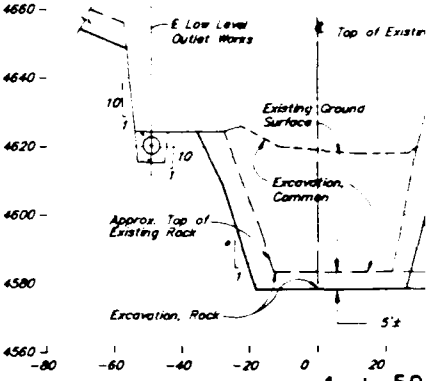
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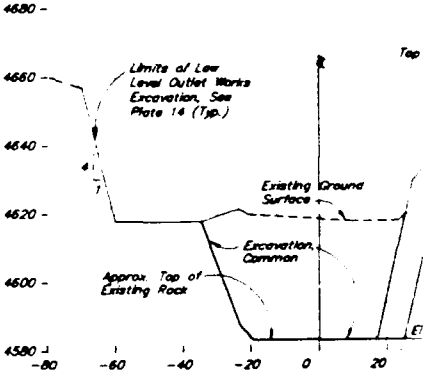
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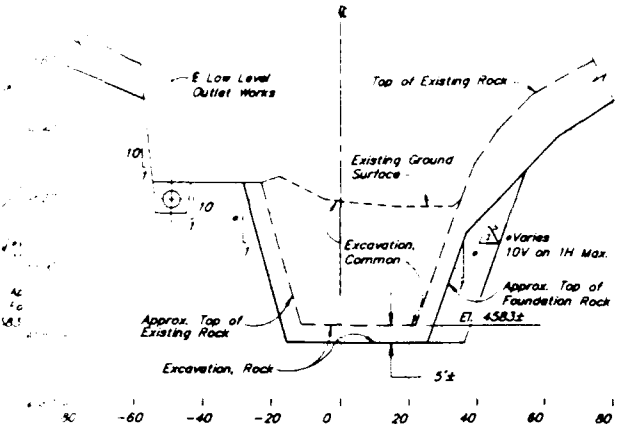
STATION 1 + 75



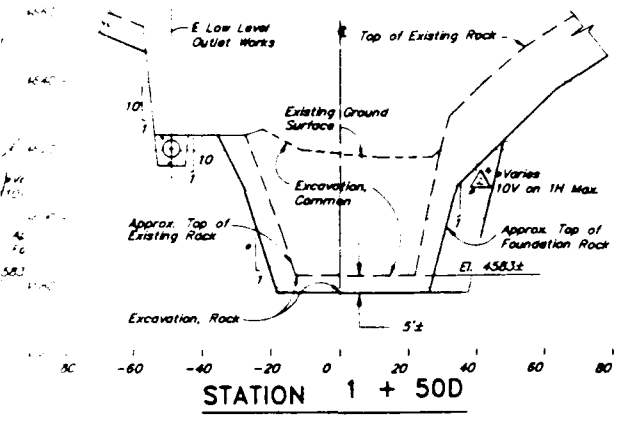
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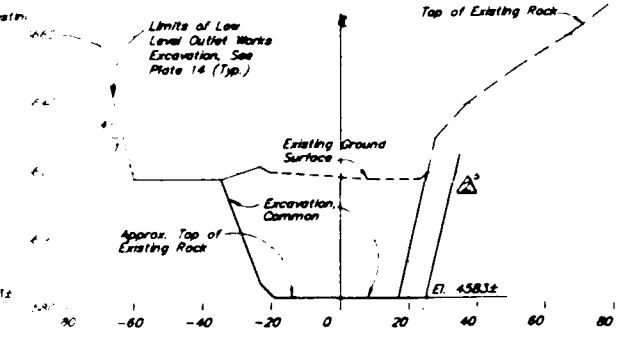
STATION 1 + 23



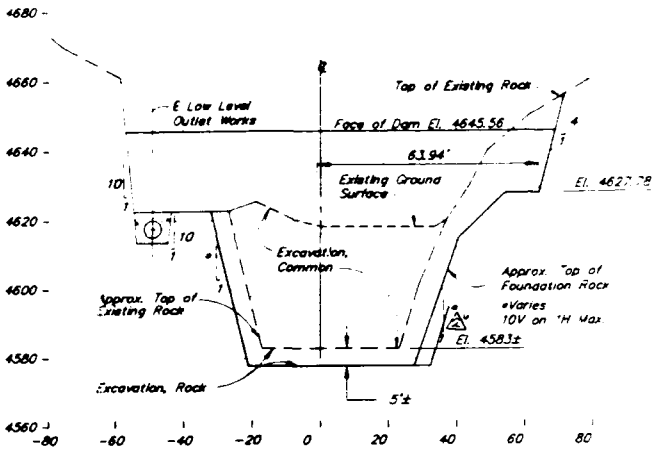
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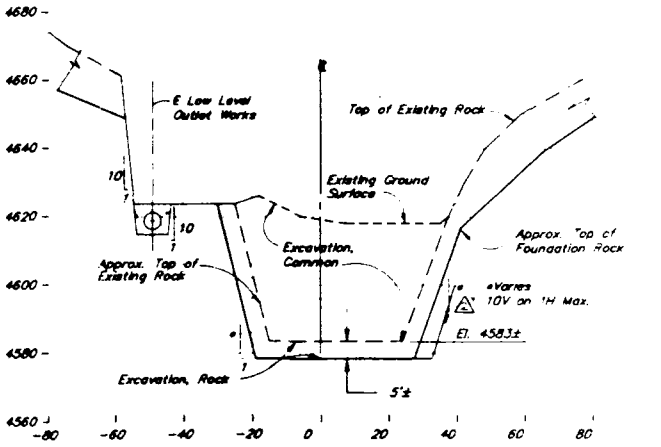
STATION 1 + 50D



STATION 1 + 23.8



STATION 2 + 25D



STATION 2 + 00D

0 20 40 60 80

Scale: 1" = 20'

⚠' All Rock Slopes Steeper Than 1V To 1H Formed By Blasting Shall Be Pre-Split

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	2+25 D&2+00 D "AS BUILT"	12/4/41	
⚠	REV STA 1+00, 2+50 D & TEL. - 1+13.9		
⚠'	ADDED NOTE (AR#2)	AUG 89	

REVISIONS

BOYLE ENGINEERING CORPORATION
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

DESIGNED BY: TWP
DRAWN BY: SBV
CHECKED BY: WMJ

RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

DAM FOUNDATION EXCAVATION SECTIONS

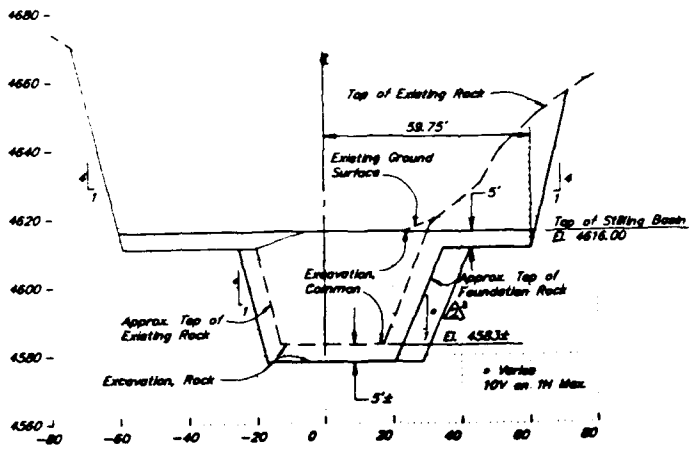
ENGINEER
DATE JULY, 1989

SOL. NO. DACW47-89-A-0241 PLATE

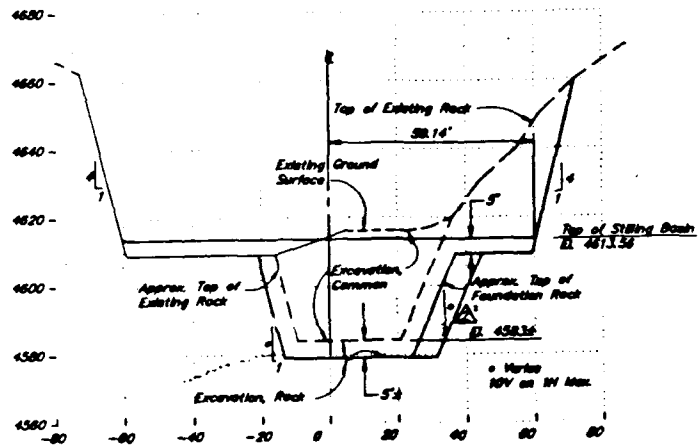
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VALUE ENGINEERING PROPOSALS MEAN HIGHER PROFITS

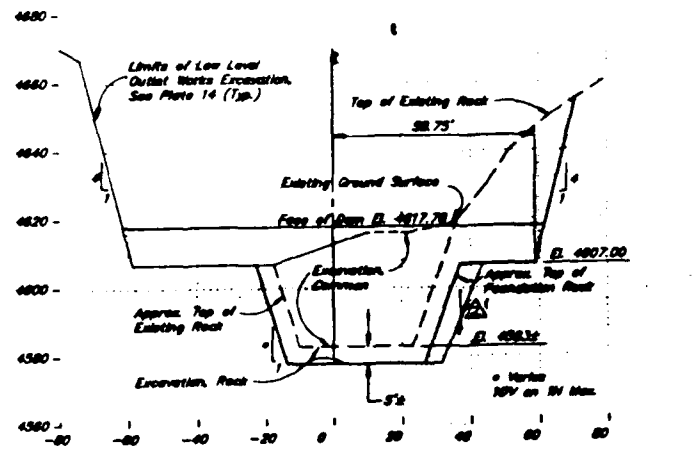
DRAWING OF WORK AS BUILT A-22



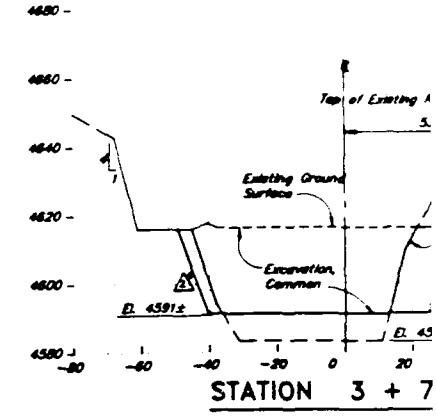
STATION 3 + 00D



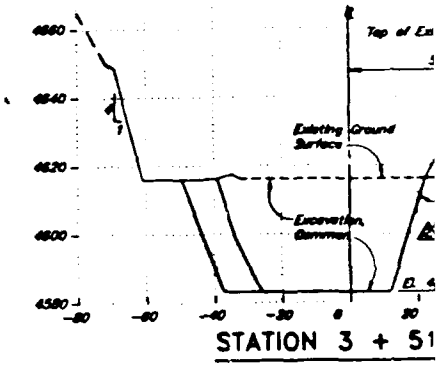
STATION 2 + 75D



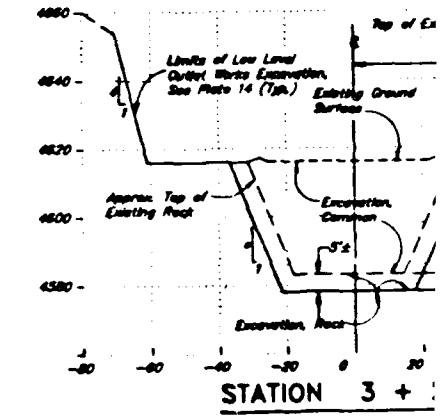
STATION 2 + 50D



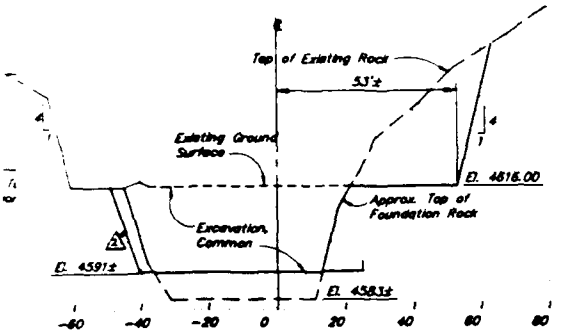
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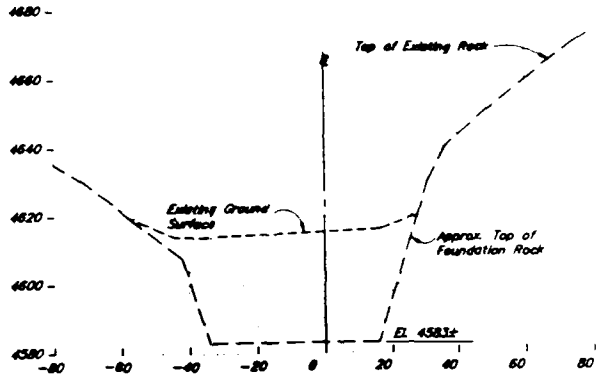
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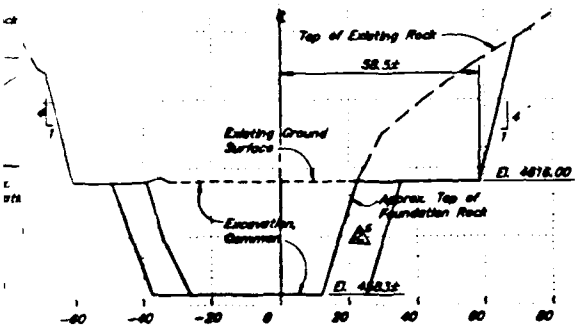
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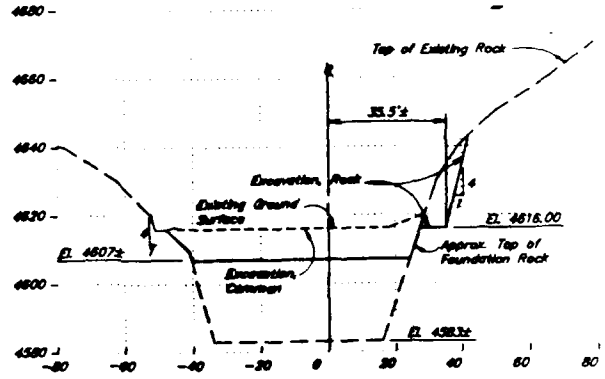
STATION 3 + 75



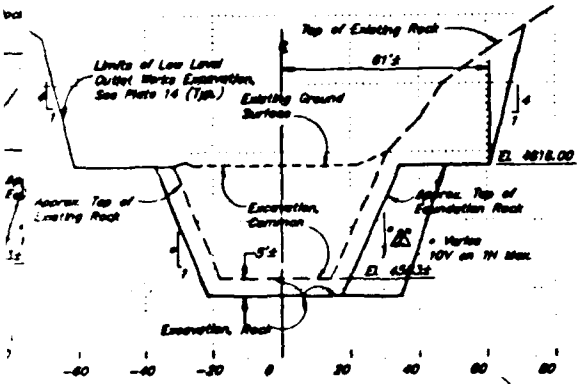
STATION 4 + 50D



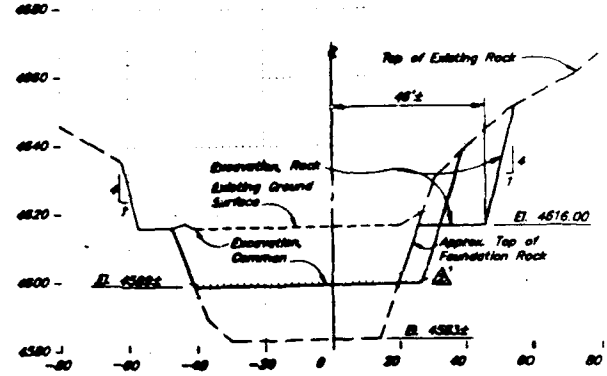
STATION 3 + 51±



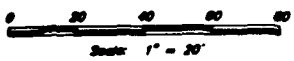
STATION 4 +25D



STATION 3 + 25D



STATION 4 + 00D



⚠1 All Rock Slopes Steeper Than 1V To 1H Formed By Blasting Shall Be Pre-Split

	4+000 "AS BUILT"	7/29/91	
⚠2	REV STA 3+00,3+25,3+50,3+75,3+800,3+850,4+000,4+250		
⚠1	ADDED NOTE (AR#2)	AUG 89	
SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

BOYLE ENGINEERING CORPORATION, CONSULTING ENGINEERS, ALBUQUERQUE, NEW MEXICO

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE, CORPS OF ENGINEERS, ALBUQUERQUE, NEW MEXICO

RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

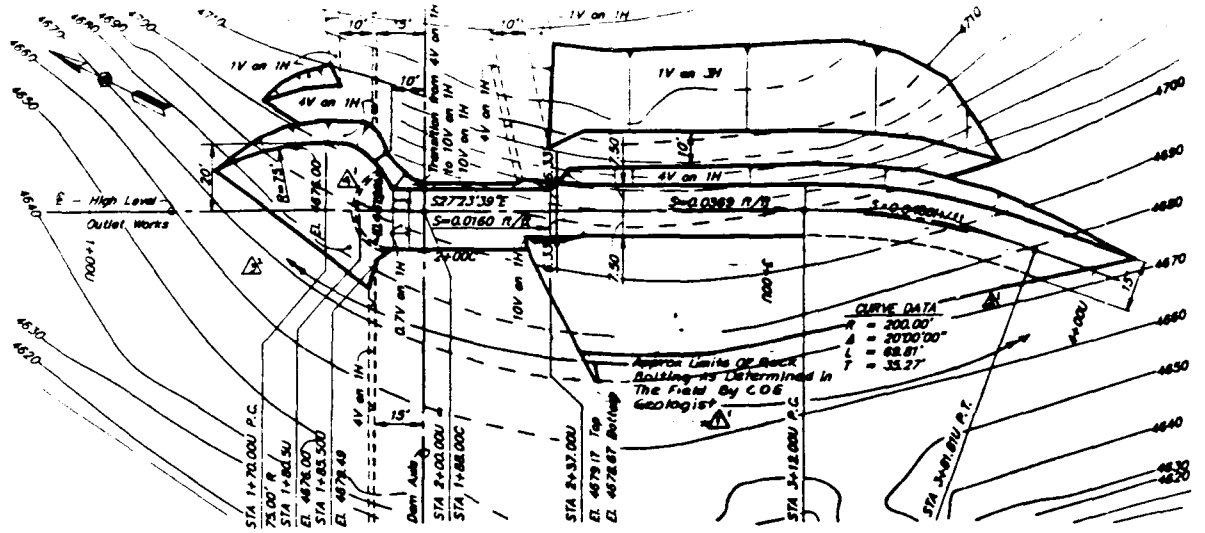
DAM FOUNDATION EXCAVATION SECTIONS

DESIGNED BY TRP
DRAWN BY SBN
TRACED BY RMJ

ENGINEER Fred Bonner
DATE JULY, 1989

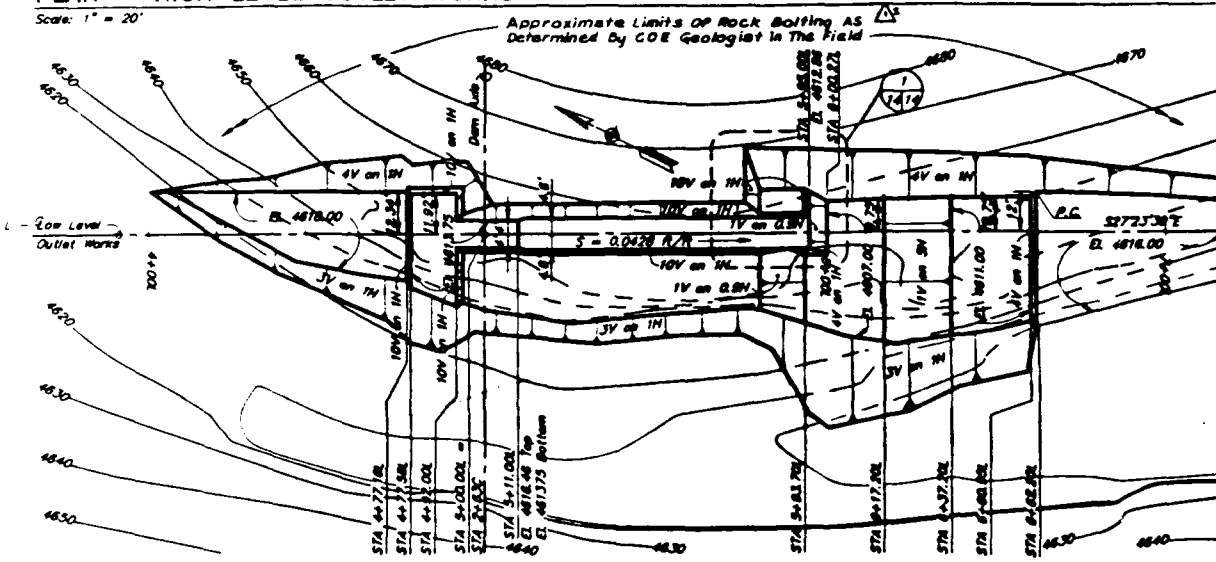
SOL.NO.DACW47-89-B-0024 PLATE A-22

FILE NUMBER RG-TC-A-12.2



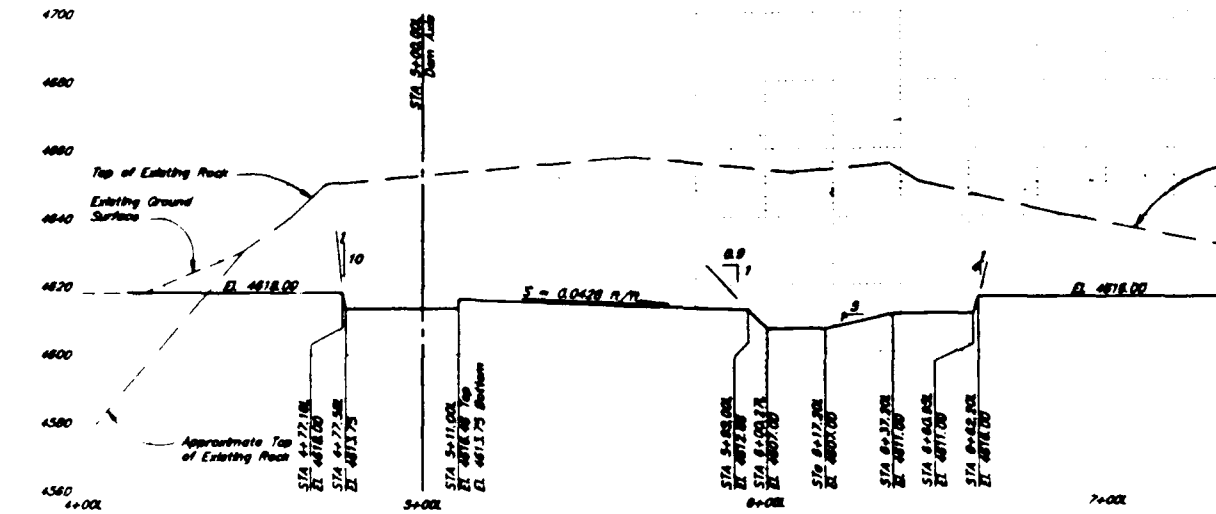
PLAN – HIGH LEVEL OUTLET WORKS

Scale: 1" = 20'



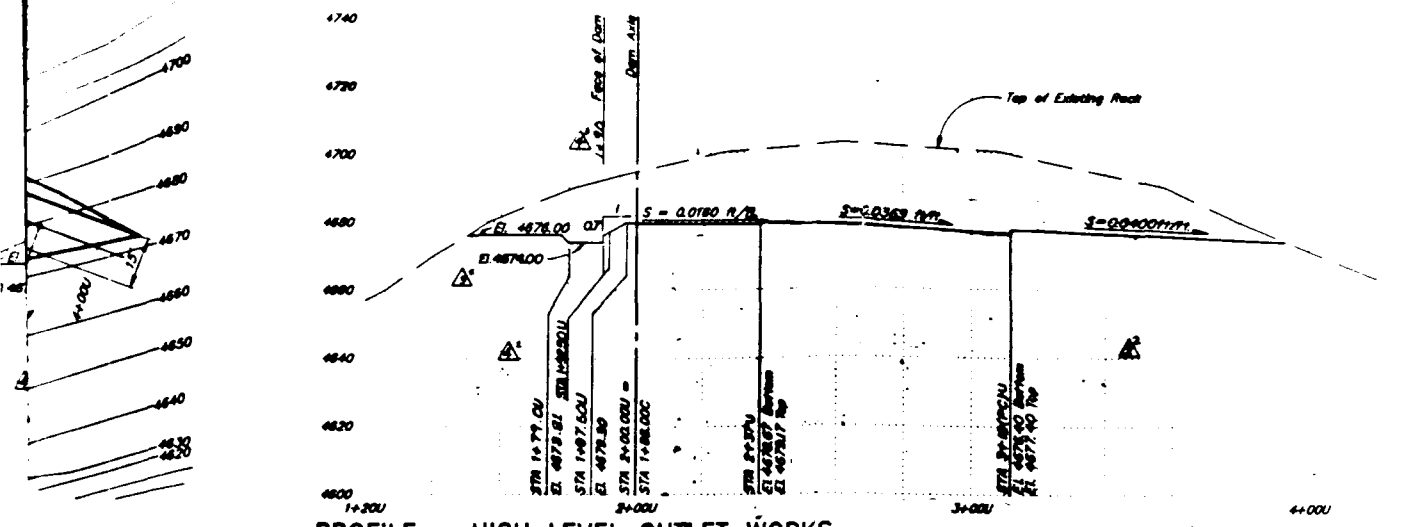
PLAN – LOW LEVEL OUTLET WORKS

Scale: 1" = 20'



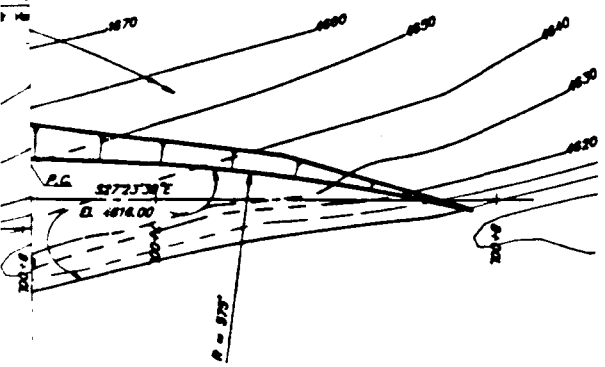
PROFILE – LOW LEVEL OUTLET WORKS

Scale: 1" = 20' Horiz & Vert

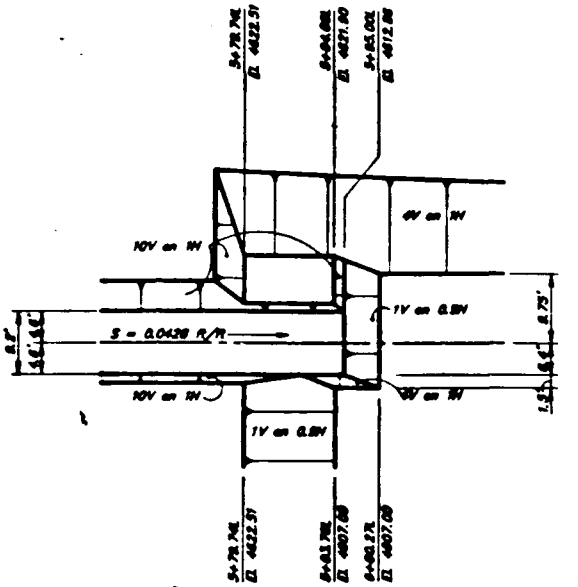


PROFILE – HIGH LEVEL OUTLET WORKS

Scale: 1" = 20'; Horiz & Vert



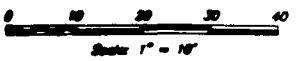




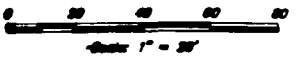
ENLARGED PLAN

Scale: 1" = 10'

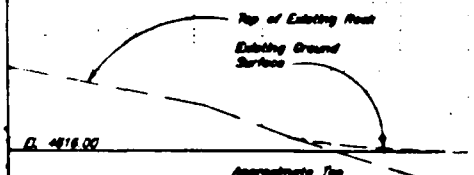
1 of 14



Scale: 1" = 10'



Scale: 1" = 20'



7+00

8+00

	"AS BUILT"		
2	REVISED HIGH LEVEL OUTLET WORKS	7/29/91	
1	REVISED HIGH LEVEL OUTLET WORKS (ML 8479)	MAY 90	
0	REVISED HIGH LEVEL OUTLET WORKS (ML 8447)	APR 90	
	ADDED NOTES (ML 8)	AUG 89	
SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

BOYLE ENGINEERING CORPORATION
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

DESIGNED BY: TBP
DRAWN BY: GEA
CHECKED BY: MMJ

RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

OUTLET WORKS EXCAVATION
PLANS & PROFILES

Fred Boyle
ENGINEER
DATE JULY, 1989

SOL.NO.DACW47-89-B-0024

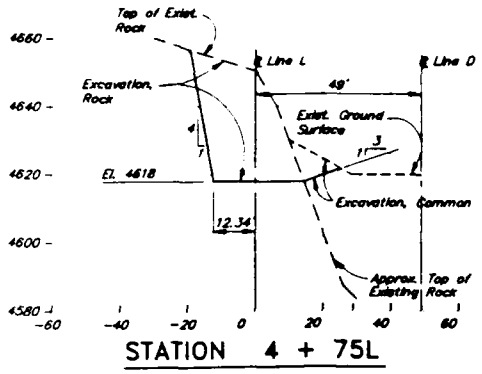
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RG-TC-A-14.4

PLATE
A-23

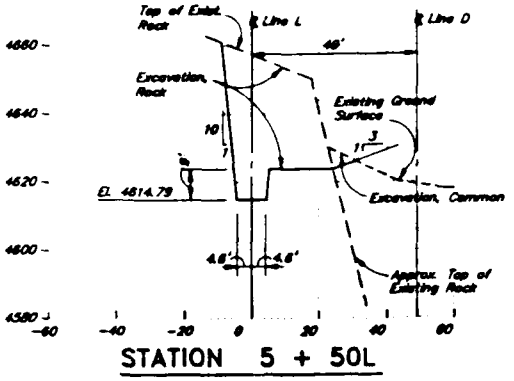
VALUE ENGINEERING PROPOSALS MEAN HIGHER PROFITS

ML 8479

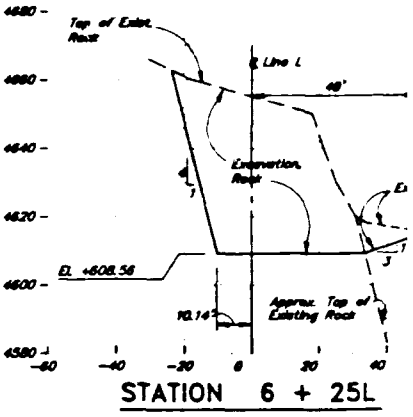
DRAWING OF WORK AS BUILT A-24



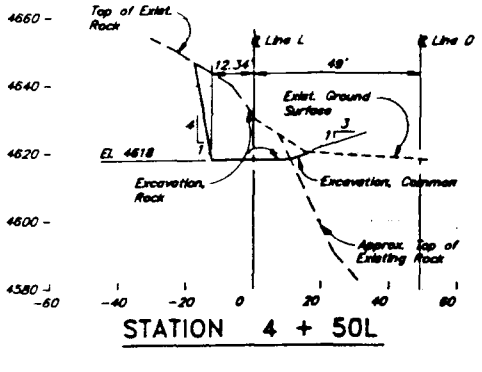
STATION 4 + 75L



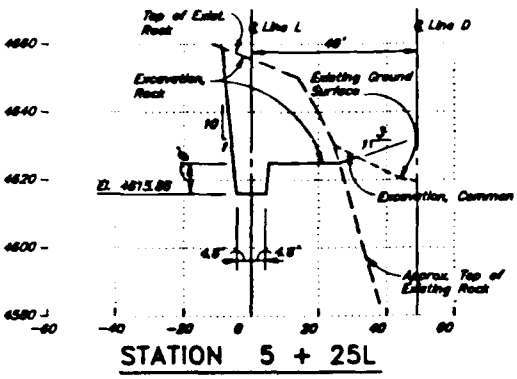
STATION 5 + 50L



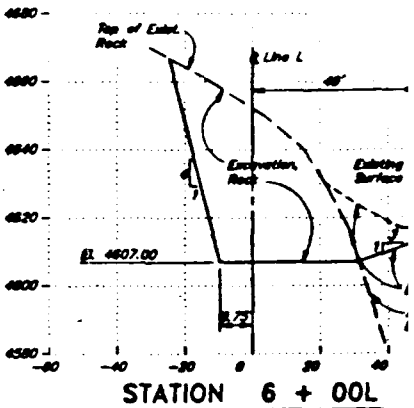
STATION 6 + 25L



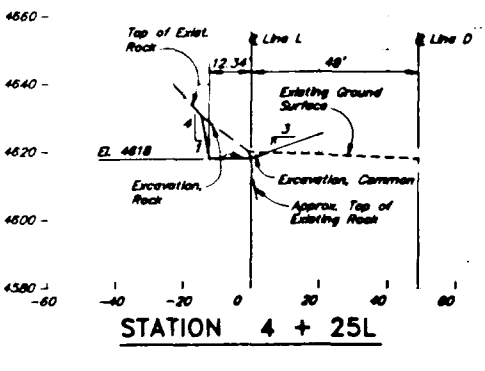
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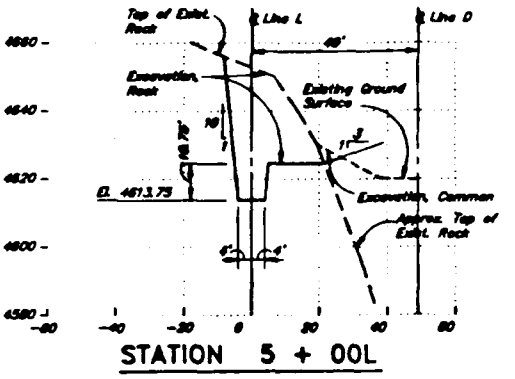
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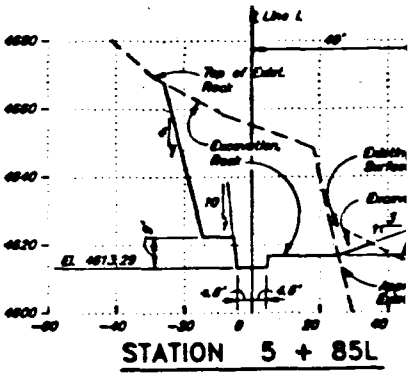
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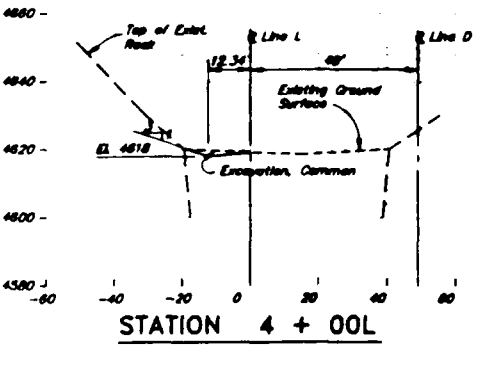
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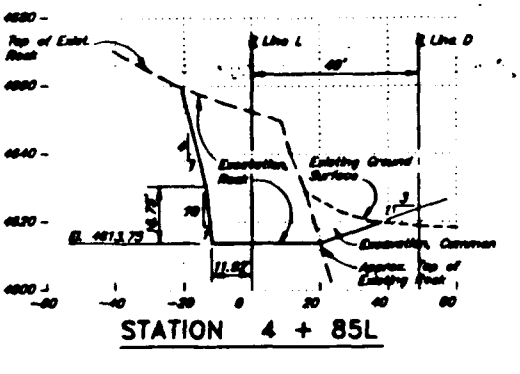
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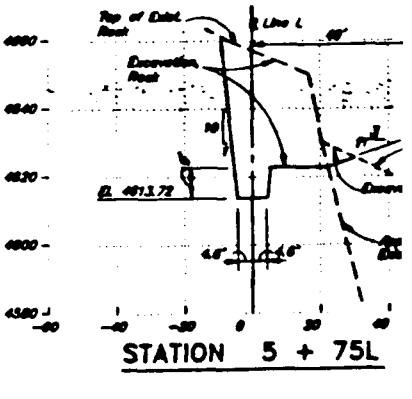
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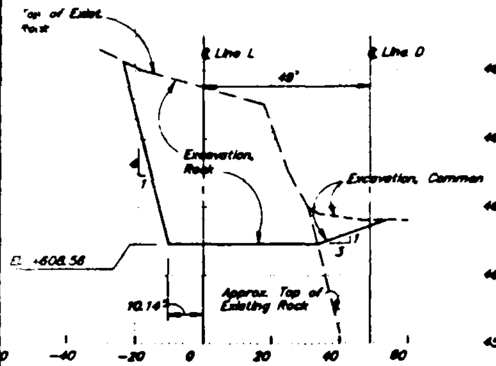
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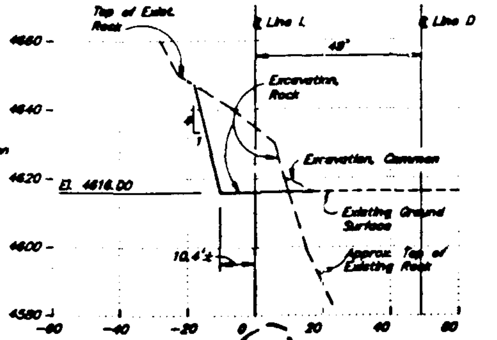
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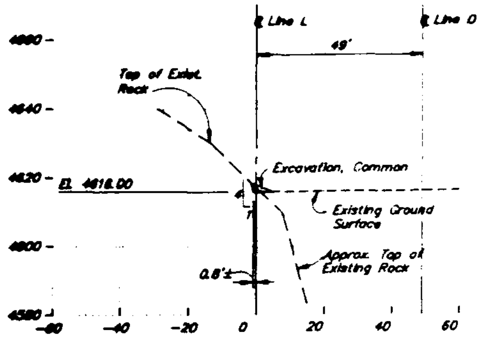
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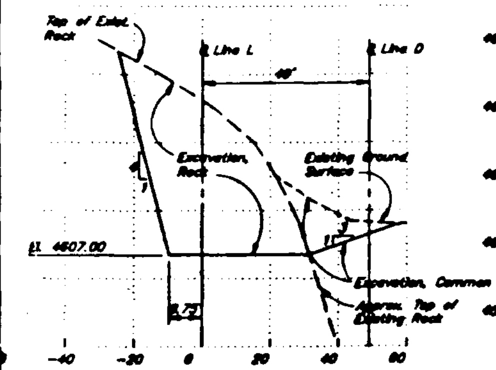
STATION 6 + 25L



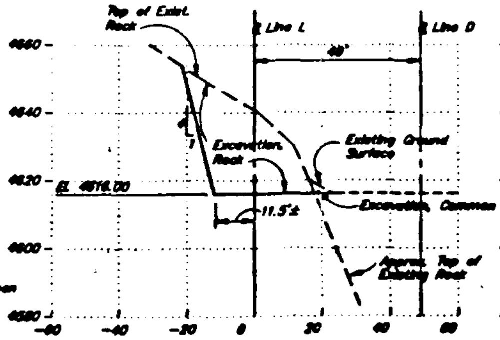
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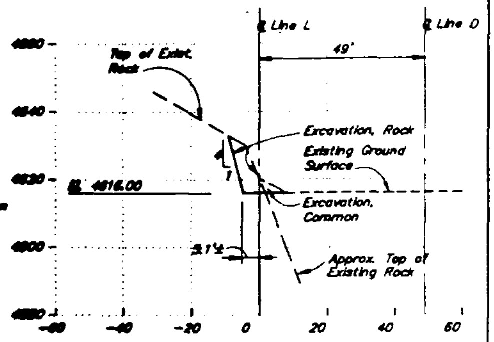
STATION 7 + 75L



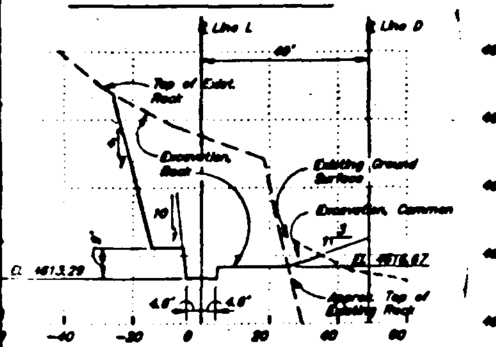
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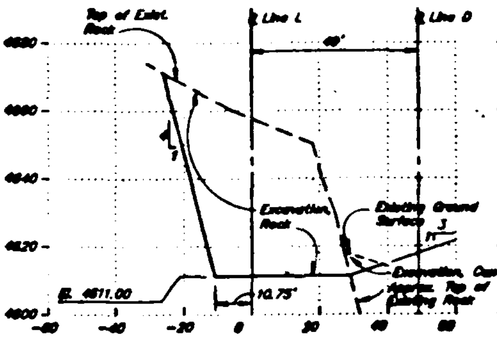
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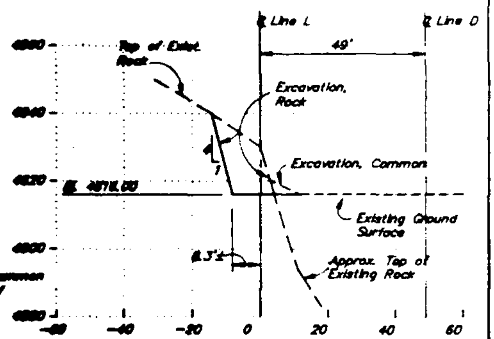
STATION 7 + 50L



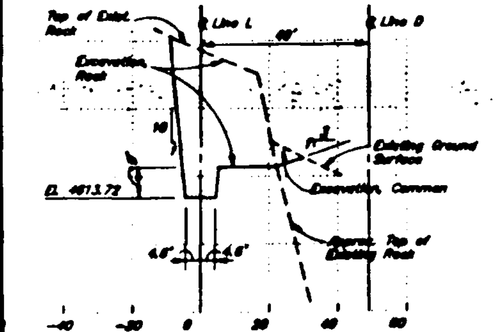
STATION 5 + 85L



STATION 6 + 50L



STATION 7 + 25L



STATION 5 + 75L

0 20 40 60 80

Scale: 1" = 20'

NOTE
All Rock Slopes Steeper Than 1V To 1H Formed By Blasting Shall Be Pre-Split

SYMBOL	DESCRIPTION	DATE	APPROVAL
1	Added Note & Revised Sta. (6+00)	Aug 89	

REVISIONS

BOYLE ENGINEERING CORPORATION
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

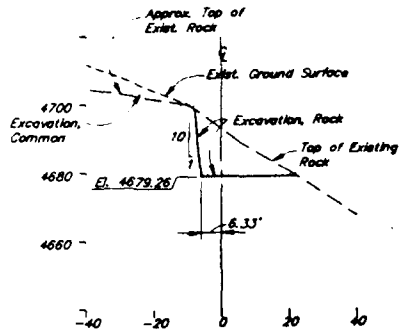
DESIGNED BY: FMP
DRAWN BY: SBN
CHECKED BY: RWJ

RIO GRANDE & TRIBUTARIES — TRUTH OR CONSEQUENCES, N.M.
RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

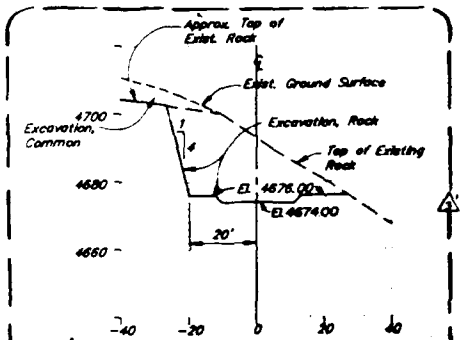
LOW LEVEL OUTLET WORKS EXCAVATION SECTIONS

SOL.NO.DACW47-89-B-0024 — PLATE A-24
FILE NUMBER RG-TC-A-15.1

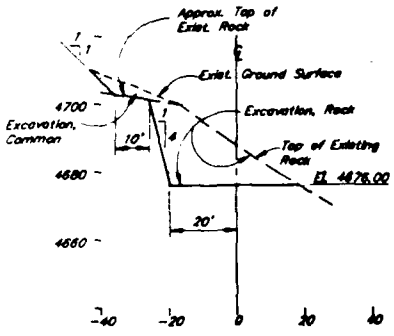
ENGINEER
DATE JULY, 1989



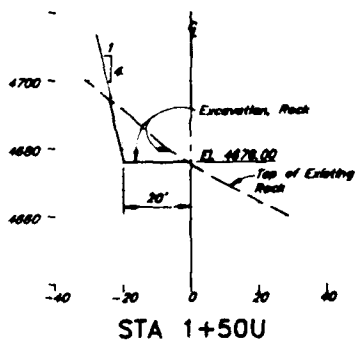
STA 2+00U



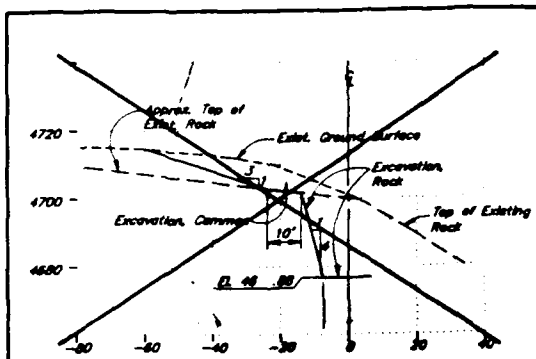
STA 1+80.5U



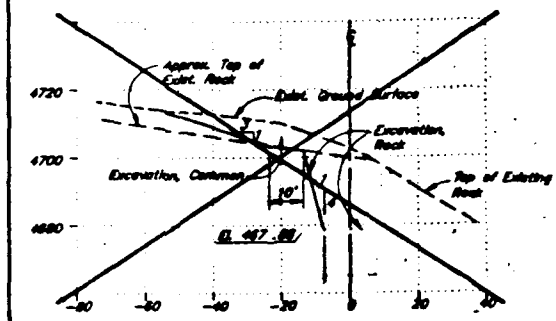
STA 1+75U



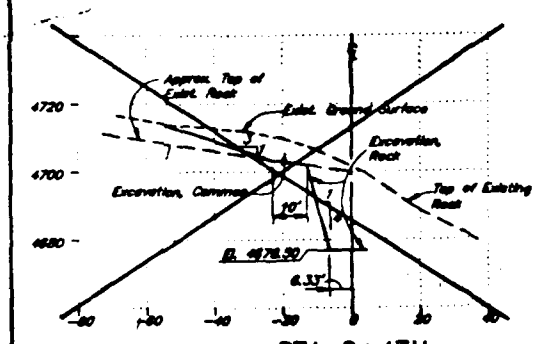
STA 1+50U



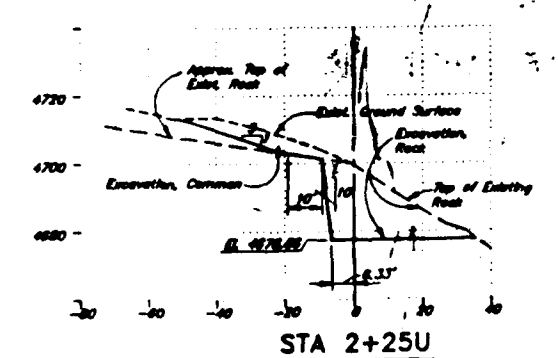
STA 3+00U



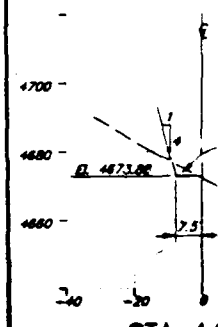
STA 2+75U



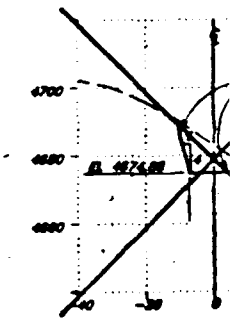
STA 2+47U



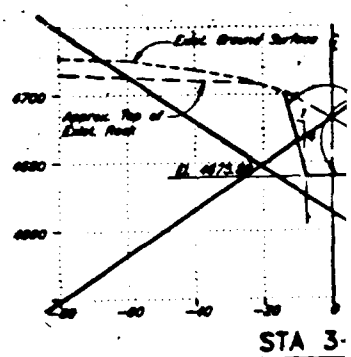
STA 2+25U



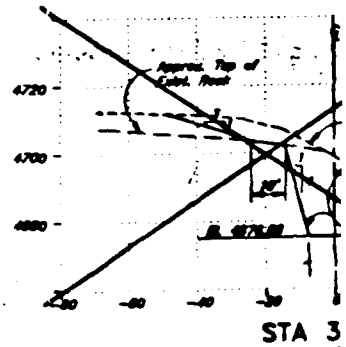
STA 4+



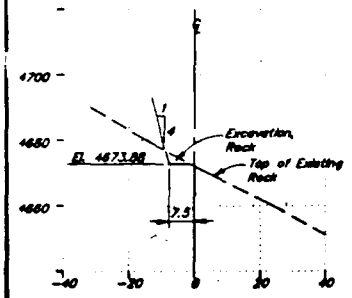
STA 3+



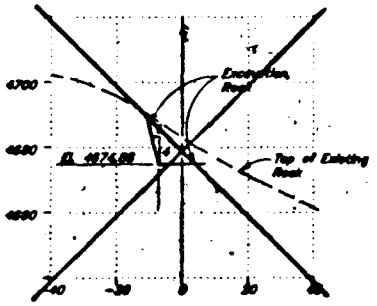
STA 3-



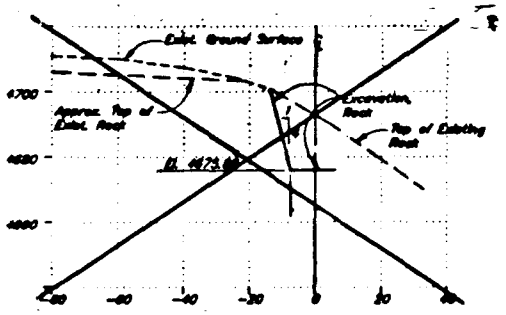
STA 3



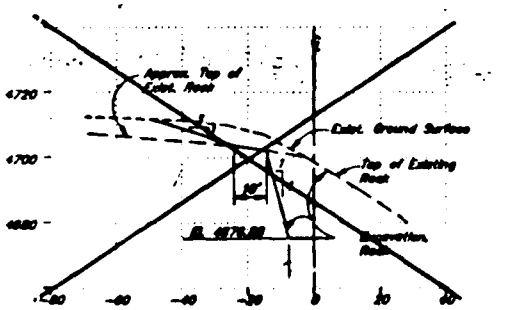
STA 4+00U



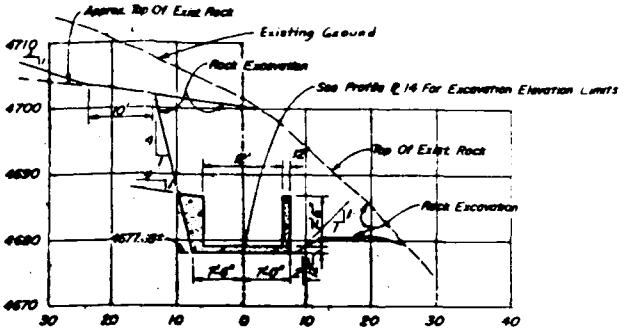
STA 3+75U



STA 3+50U



STA 3+25U



SECTION 8 STA 2+37U to 3+12U
SCALE: 1"=10' (SIMILAR)



NOTE
All Rock Slopes Steeper Than 1V To 1H Formed By Blasting Shall Be Pre-Split.

2	Revised Excavation - Sta. 1+00 to 5+40 (ML 9479) per 90	May 90	
1	3rd TSU Added Sheet 8 (ML 9447)		
	Deleted Stas 2+88, 3+87.5, 3+00U, 3+25U, 3+50U, & 3+75U per 90		
	Added Note in No. 2	Aug 89	
SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

BOYLE ENGINEERING CORPORATION CONSULTING ENGINEERS ALBUQUERQUE, NEW MEXICO	U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE CORPS OF ENGINEERS ALBUQUERQUE, NEW MEXICO

DESIGNED BY TMP
DRAWN BY SBH
CHECKED BY MMJ

RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

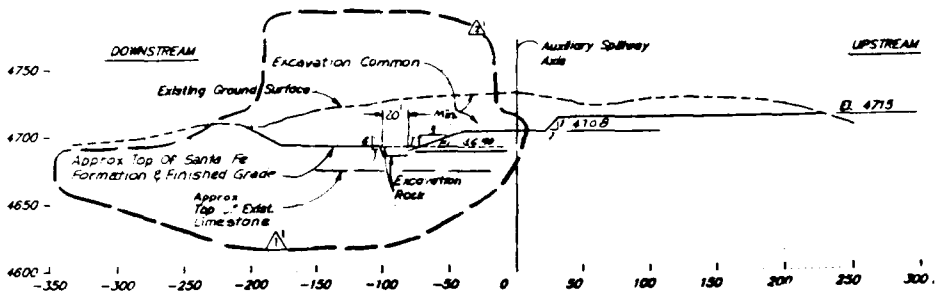
HIGH LEVEL OUTLET WORKS EXCAVATION SECTIONS

ENGINEER
DATE JULY, 1989

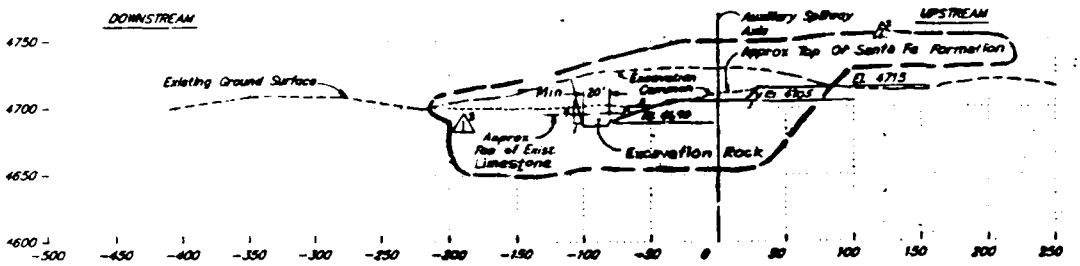
SOL.NO.DACW47-89-B-0024 PLATE
FILE NUMBER RG-TC-A-163 A-25

VALUE ENGINEERING PROPOSALS MEAN HIGHER PROFITS

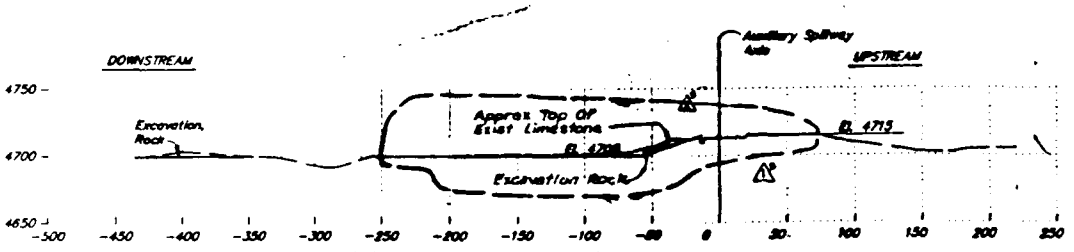
DRAWING OF WORK AS BUILT ML 9479 A-26



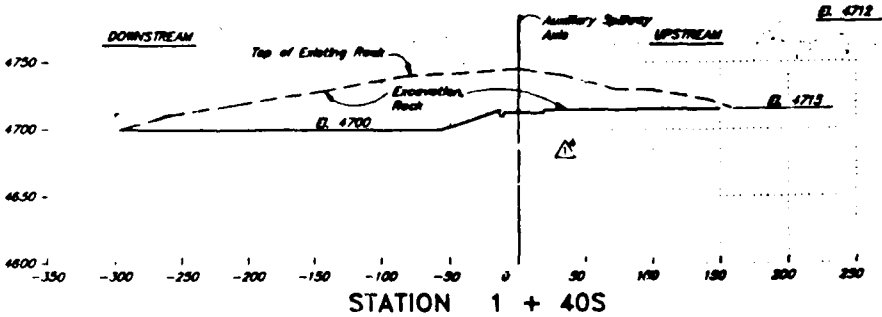
STATION 3 + 00S



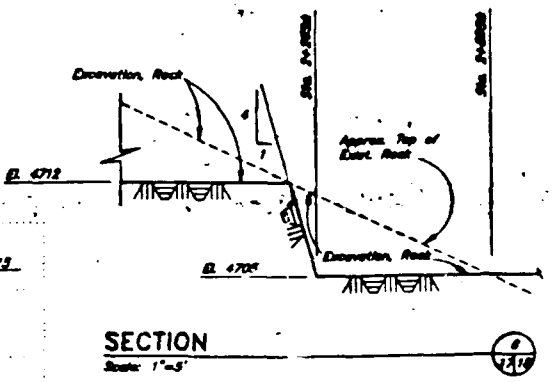
STATION 2 + 50S



STATION 2 + 00S

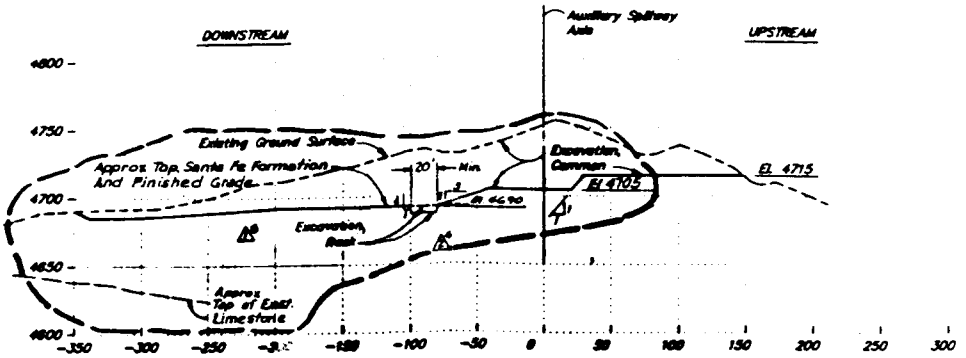


STATION 1 + 40S

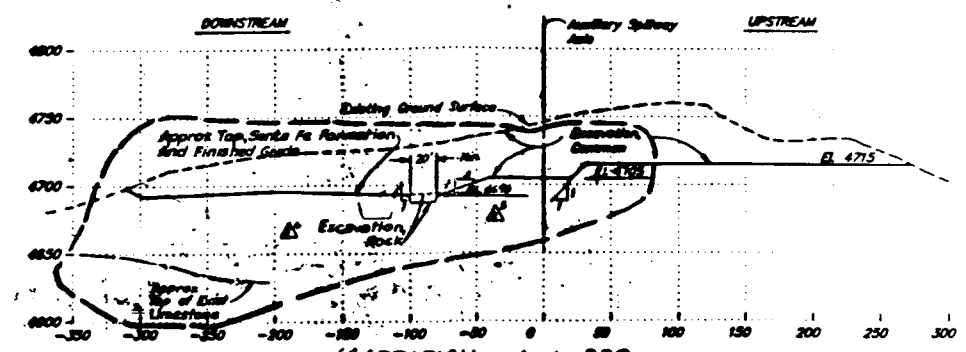


SECTION

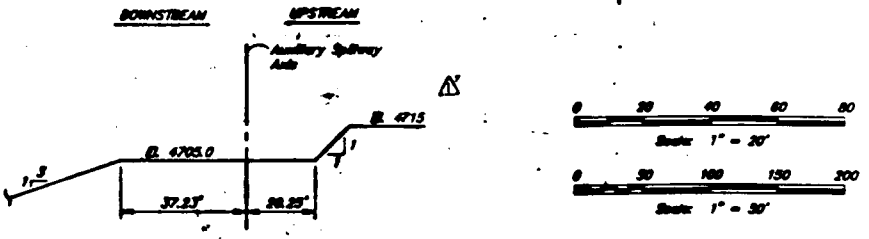
Scale: 1"=5'



STATION 5 + 00S

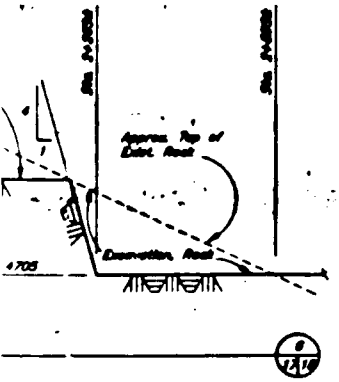


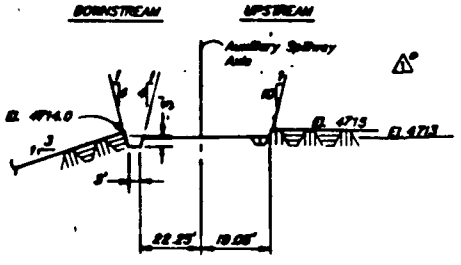
STATION 4 + 00S



STA. 2+26S± to STA. 8+20S
CREST IN OVERBURDEN DETAIL

Scale: 1" = 20'





STA. 1+40S to 2+26S±
EXCAVATION AT CREST IN ROCK DETAIL

Scale: 1" = 20'

	El. 4+00S (TM 8441)		
2	Revised Sections 3+00S, 2+30S, 2+00S, 5+00S		
1	Revised Sections 3+00S, 2+30S, 2+00S, 5+00S (Dwg. #2)	Aug. 89	
SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

BOYLE ENGINEERING CORPORATION CONSULTING ENGINEERS ALBUQUERQUE, NEW MEXICO	U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE CORPS OF ENGINEERS ALBUQUERQUE, NEW MEXICO

DESIGNED BY: FRE
DRAWN BY: SBM
CHECKED BY: RMJ

RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

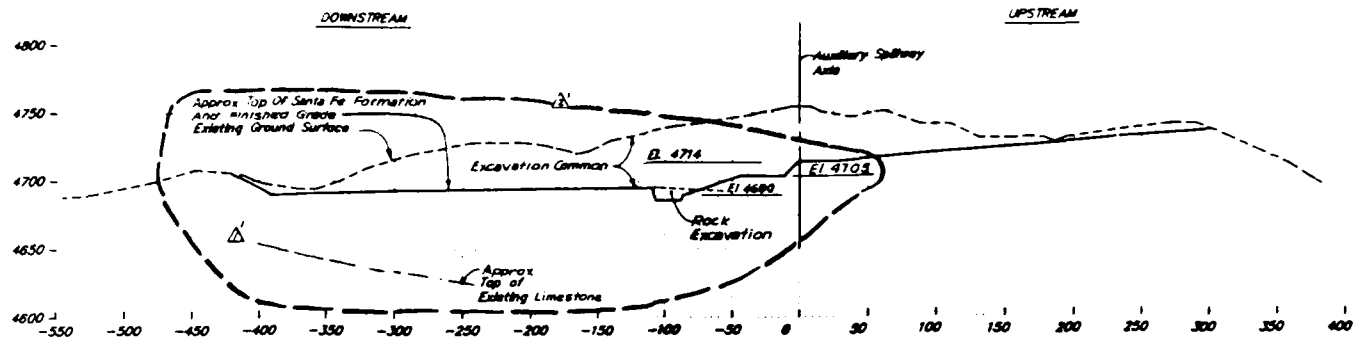
AUXILIARY SPILLWAY FOUNDATION EXCAVATION SECTIONS & DETAILS

Fred Bamm
ENGINEER
DATE JULY, 1989

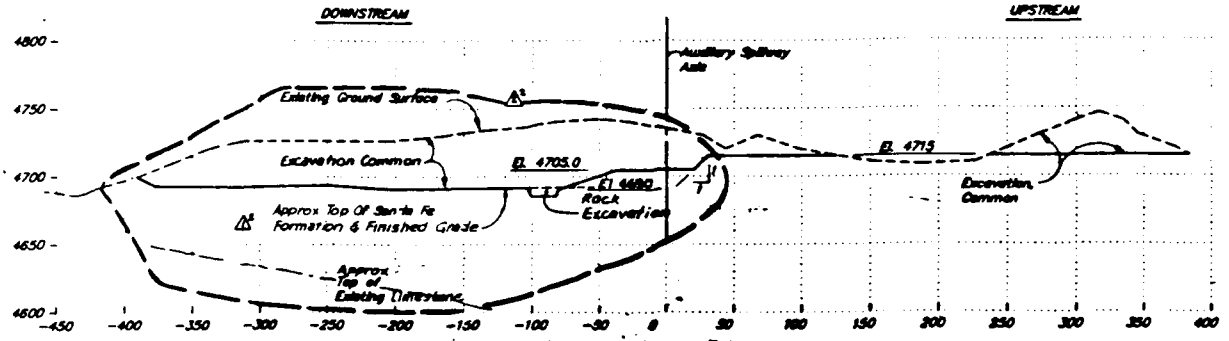
SOL.NO.DACW47-89-B-0024
FILE NUMBER RG-TC-A-18.2
PLATE A-26

VALUE ENGINEERING PROPOSALS MEAN HIGHER PROFITS

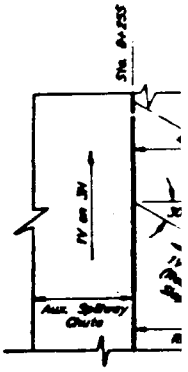
DRAWING OF WORK AS BUILT ML 8441A-27



STATION 8 + 20S

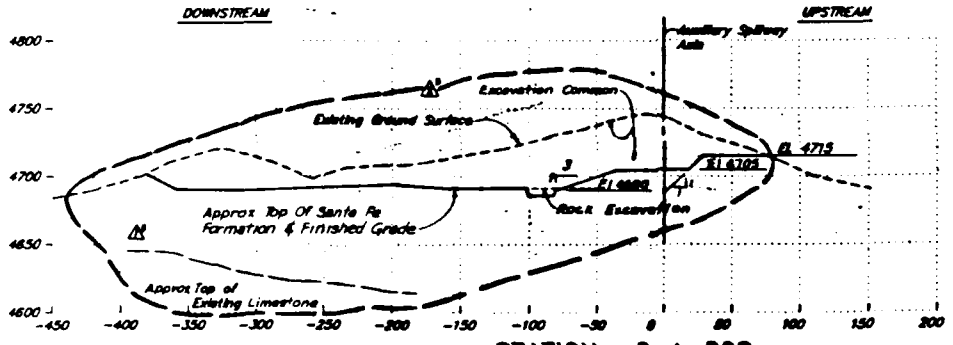


STATION 7 + 50S

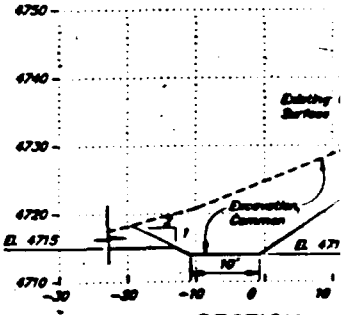


DETAIL A

N.T.S.

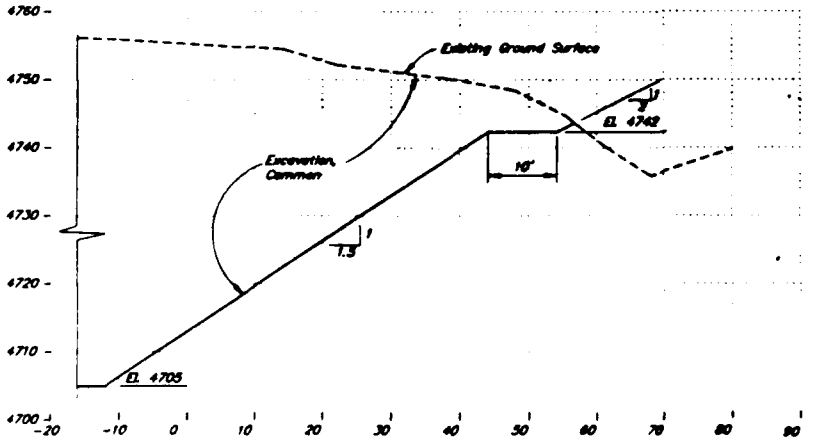


STATION 6 + 00S



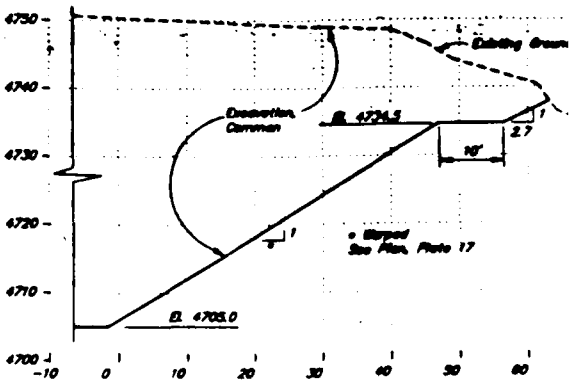
SECTION

Scale: 1" = 10'



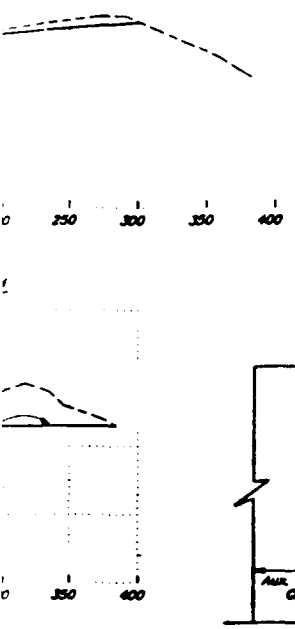
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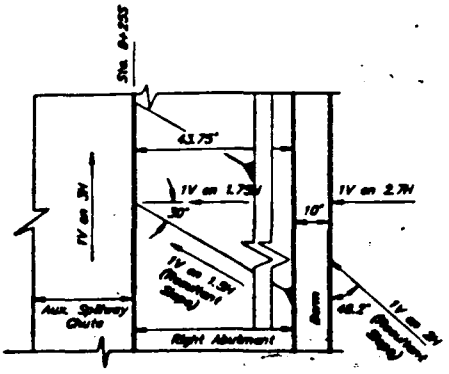
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SECTION

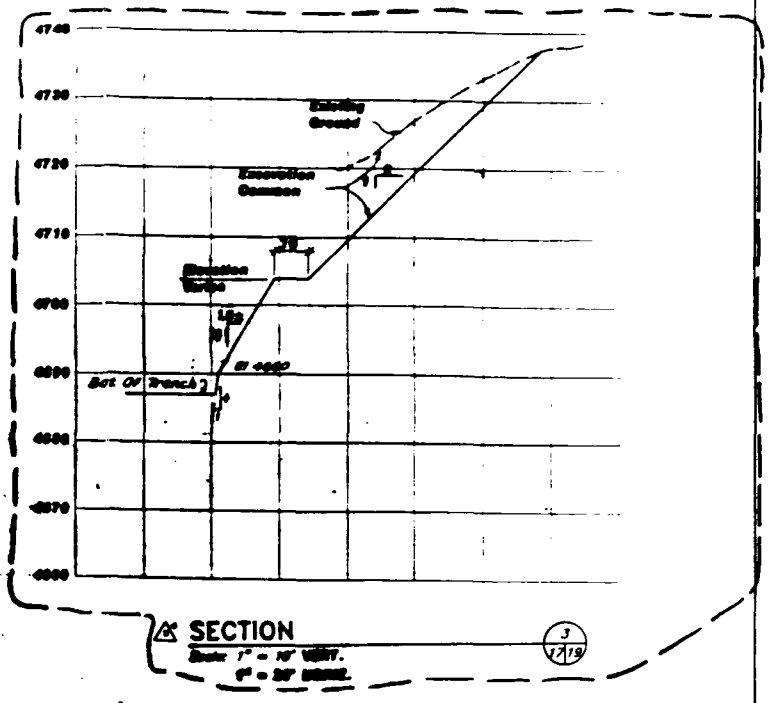
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DETAIL A

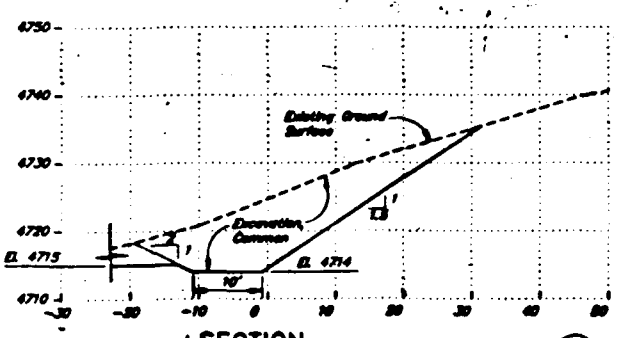
M.T.S.



SECTION

Scale 1" = 10' VERT.
1" = 20' HORIZ.

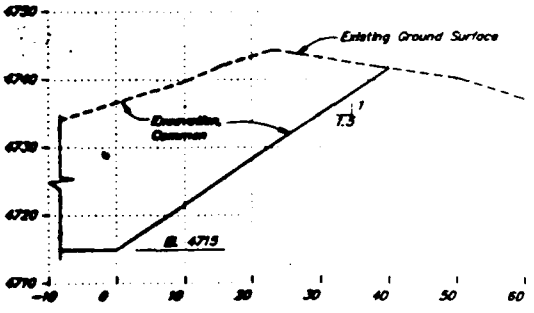
3
17/19



SECTION

Scale 1" = 10'

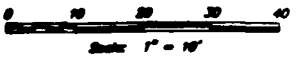
4
17/19



SECTION

Scale 1" = 10'

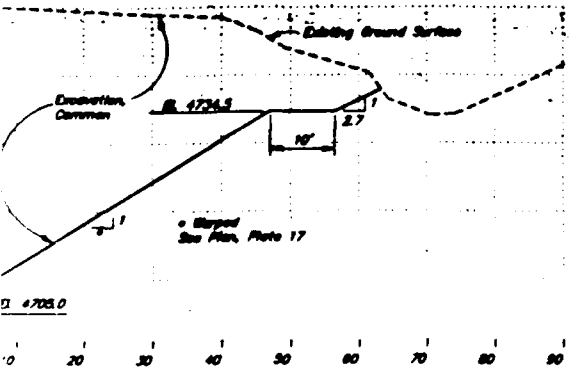
5
17/19



Scale 1" = 10'



Scale 1" = 50'



SECTION

Scale 1" = 10'

2
17/19

2	Revised Sections (BL 8441)	Mar 90	
1	Changed Excavation & Rock Line (AR82)	Aug 89	
SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

BOYLE ENGINEERING CORPORATION
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

DESIGNED BY: PRL
DRAWN BY: SBN
CHECKED BY: RMJ

RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

AUXILIARY SPILLWAY FOUNDATION EXCAVATION SECTIONS & DETAIL

Fred Barnes
ENGINEER
Date JULY, 1989

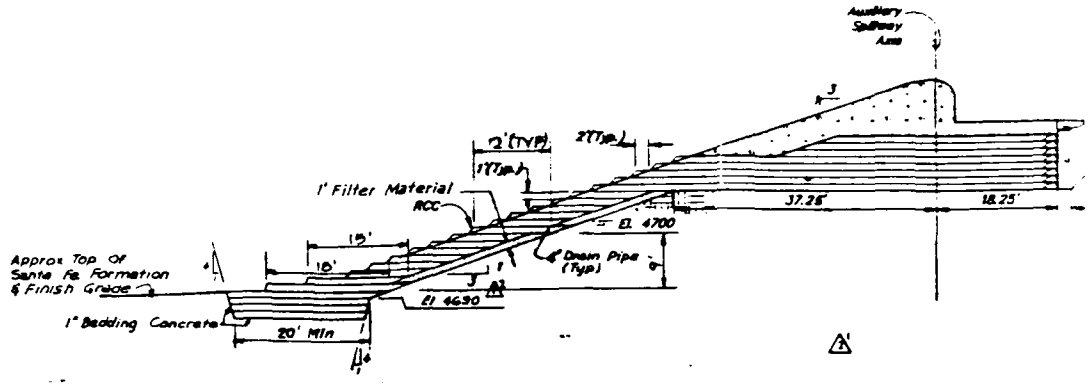
SOL.NO.DACW47-89-B-0024

FILE NUMBER
RG-TC-A-19.2

PLATE
A-27

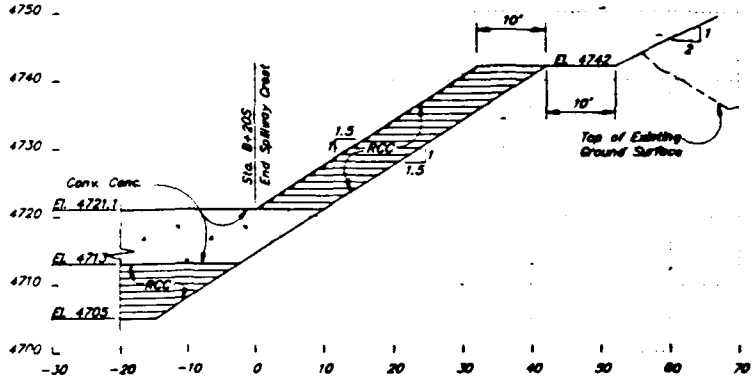
VALUE ENGINEERING PROPOSALS MEAN HIGHER PROFITS

DRAWING OF WORK AS BUILT ML 8441 A-28



STA 2+26S± to STA 8+20S
TYPICAL SECTION

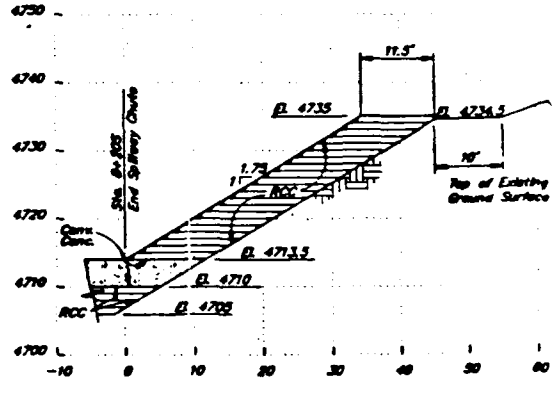
Scale: 1"=10'



SECTION

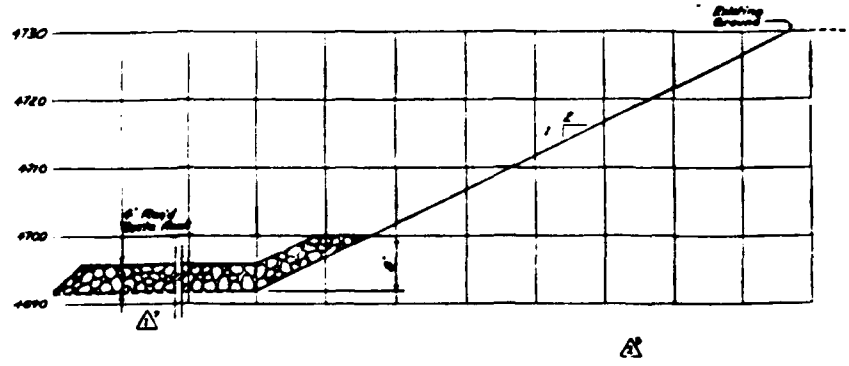
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1/20,21



SECTION

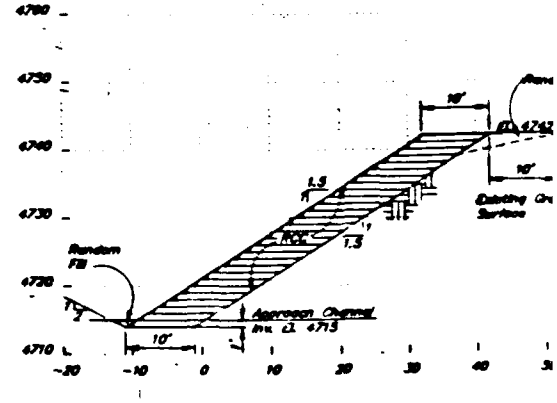
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SECTION

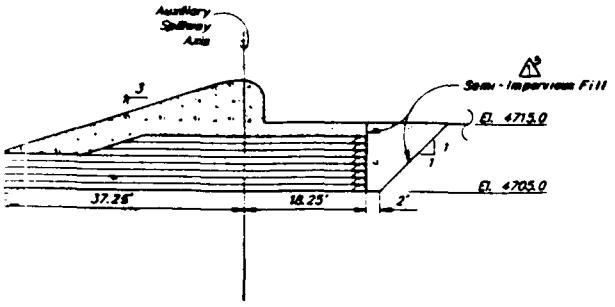
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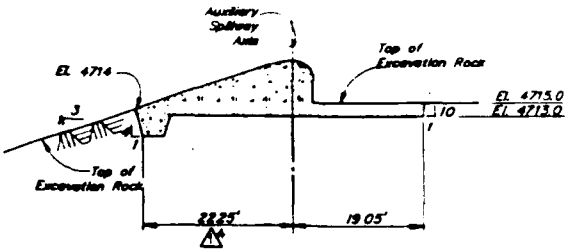
3/20,21



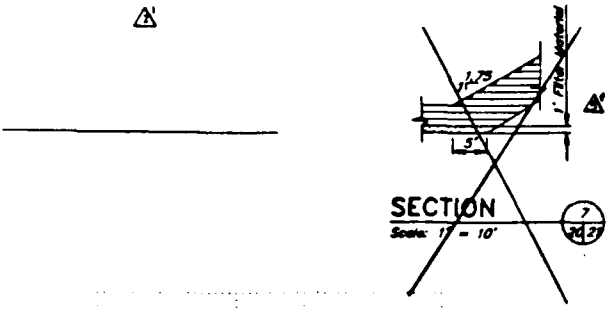
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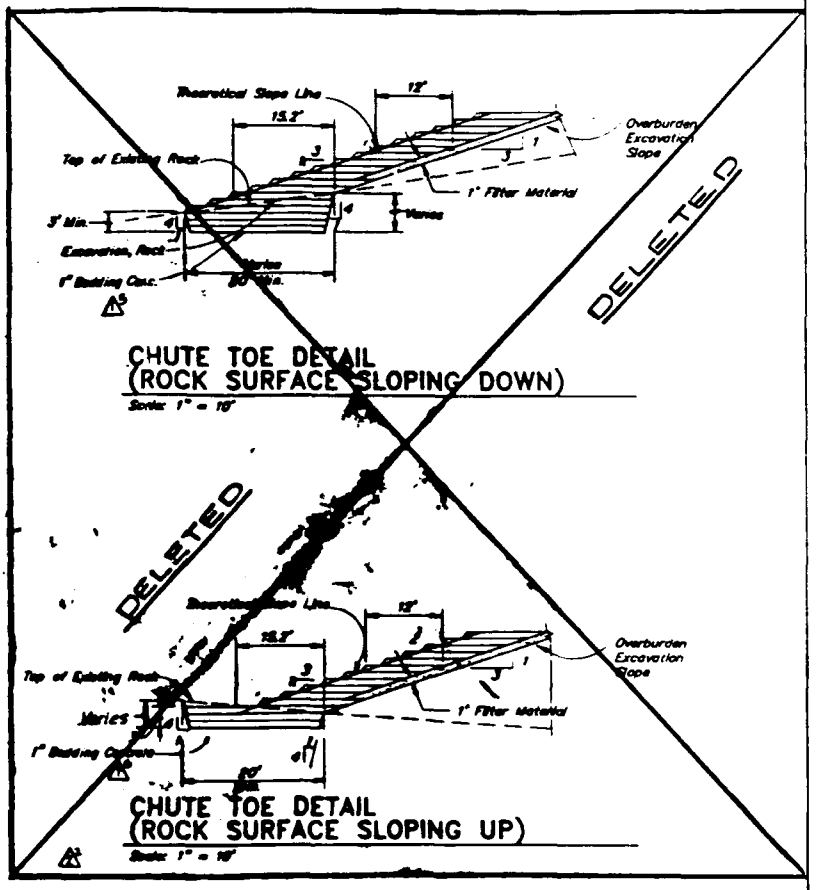
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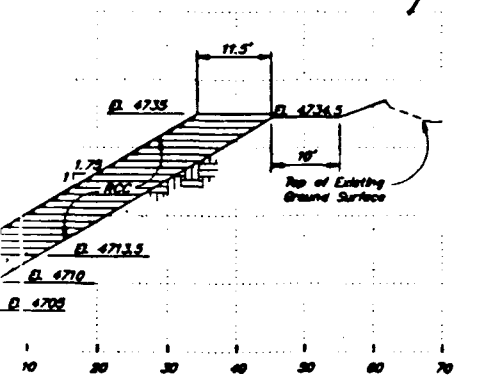


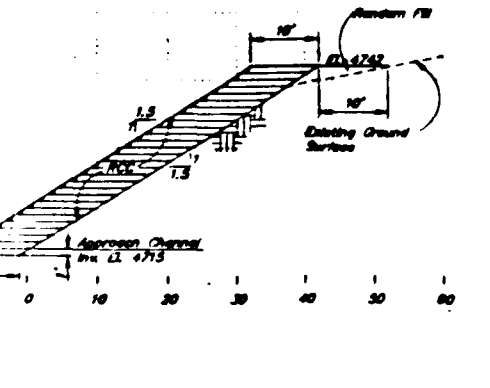


STA 1+40S to 2+26S±
TYPICAL SECTION
Scale: 1" = 10'









0 10 20 30 40
Scale: 1" = 10'

Note: See Plate 30 For 4" ϕ Drain Pipe.

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		
1	DISREGARD AS PER RFI # 25 "AS BUILT"	7/29/91	
2	REVISED TYP. SECT'S. (ML 9441)	MAR 1992	
3	ADDED NOTES, REVISED TYP. SECTS. & CHUTE TOE DETS. (SHEET 2)	AUG 1992	

BOYLE ENGINEERING CORPORATION CONSULTING ENGINEERS ALBUQUERQUE, NEW MEXICO	U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE CORPS OF ENGINEERS ALBUQUERQUE, NEW MEXICO

DESIGNED BY: PWL
DRAWN BY: SBW
CHECKED BY: RWJ

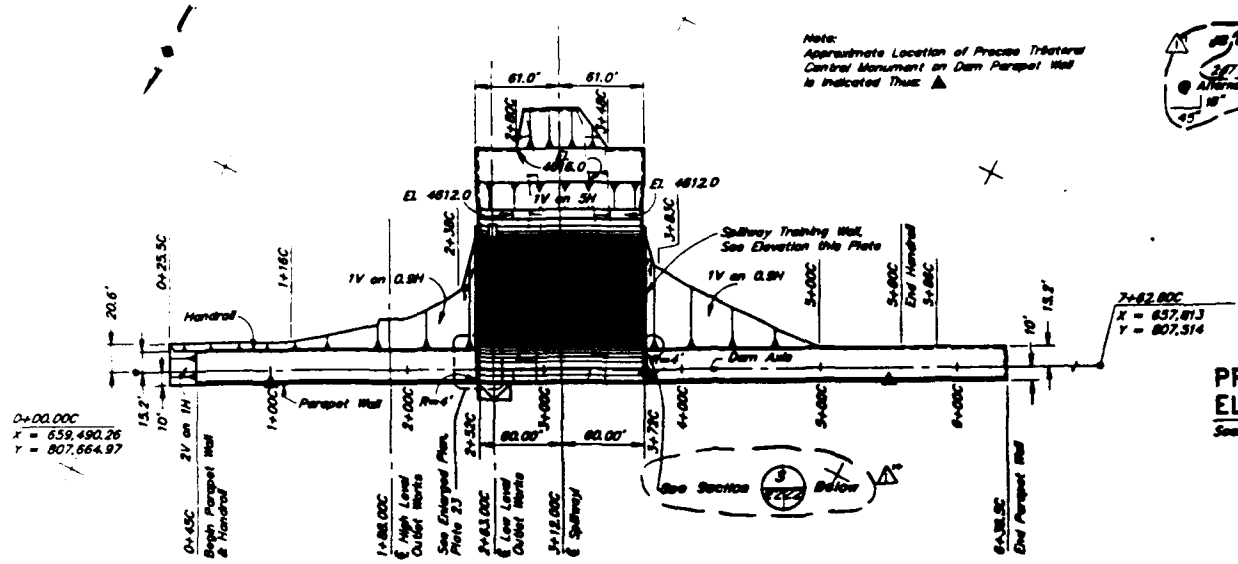
RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

AUXILIARY SPILLWAY GRADING SECTIONS

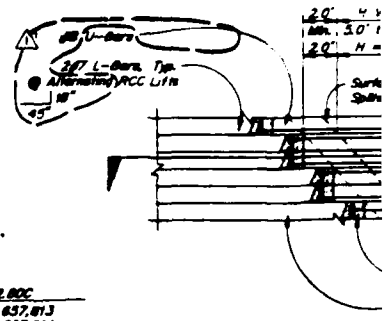
Fred Brown
ENGINEER
DATE JULY 1989

SOL.NO.DACW47-89-B-0024 PLATE
FILE NUMBER RG-TC-A-21.3 A-28



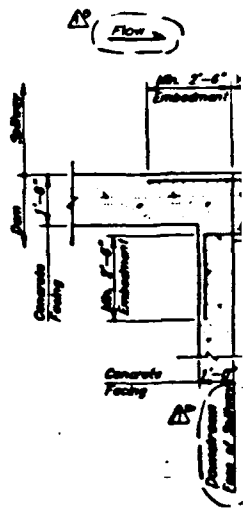
STRUCTURAL PLAN

Scale: 1"=50'



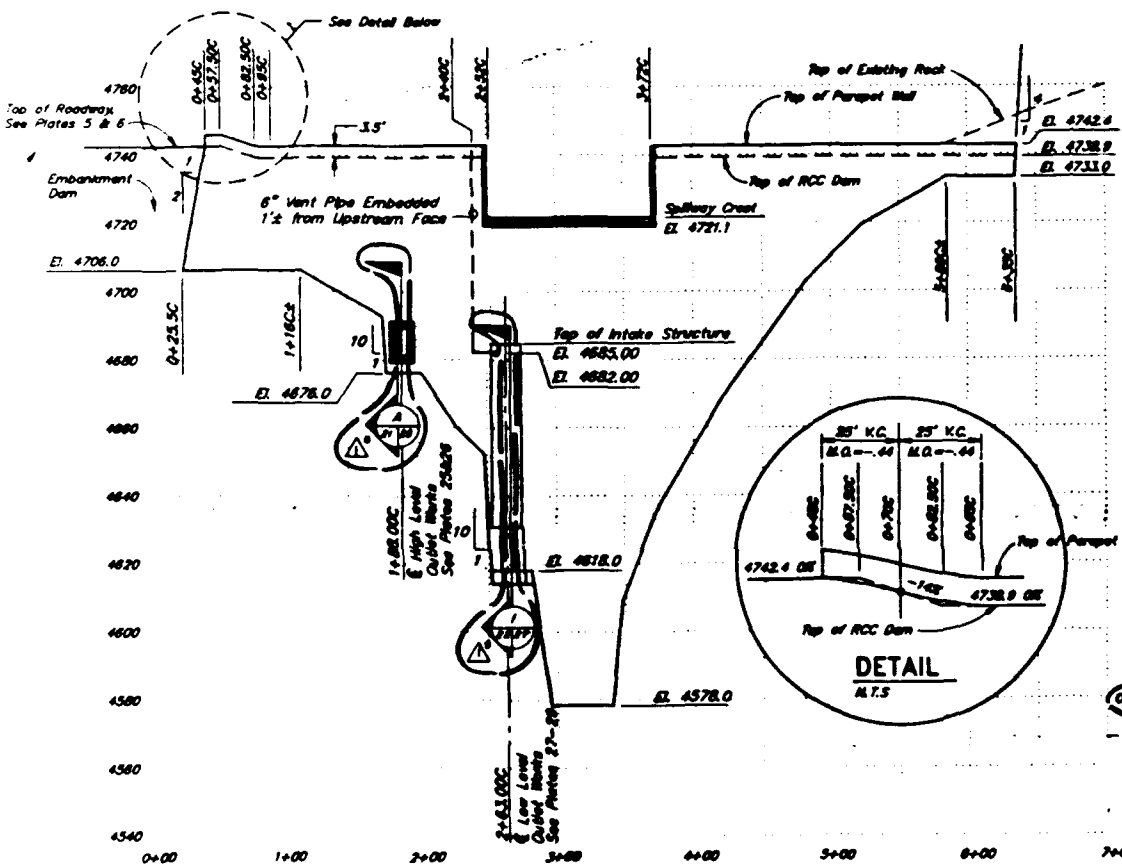
PRECAST CONCRETE TRA
ELEVATION

Scale: 1"=4'



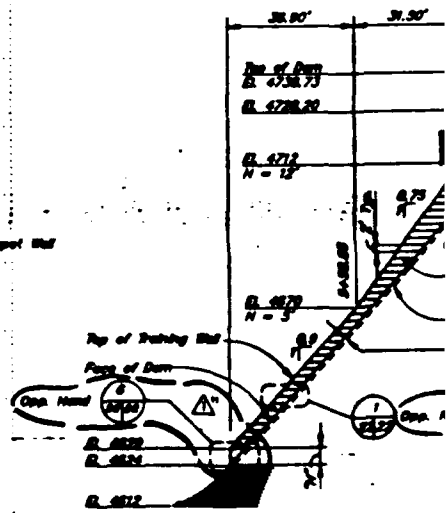
CAST-IN-PLACE
CONCRETE TRAIN

Scale: 1" = 5'



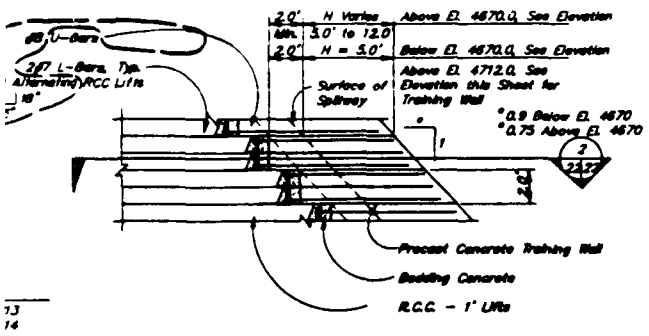
ELEVATION LOOKING DOWNSTREAM

Scale: 1"=50', Horiz; 1"=20', Vert



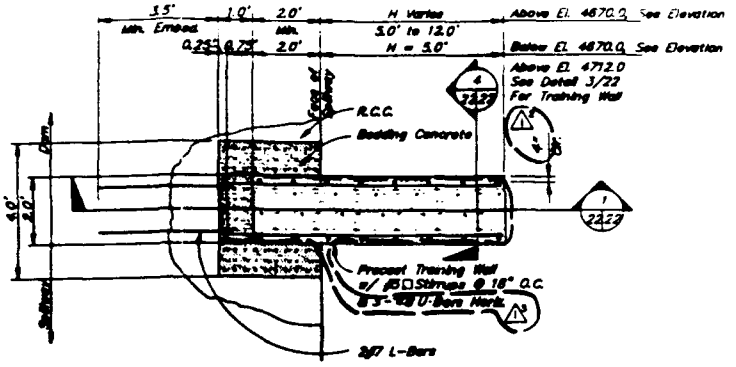
TRAINING WALL E

Scale: 1" = 20'; Horiz & Vert



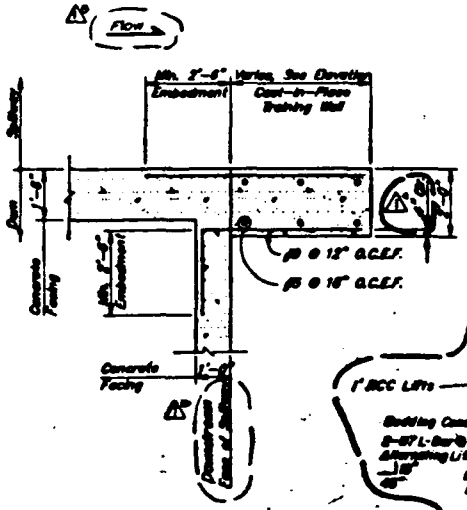
PRECAST CONCRETE TRAINING WALL ELEVATION

Scale: 1"=4'



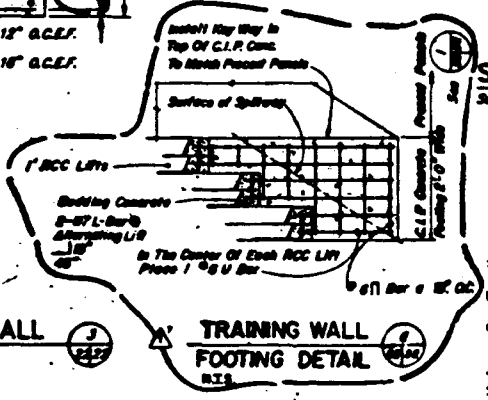
PRECAST CONCRETE TRAINING WALL REINFORCING - PLAN

Scale: 1"=3'



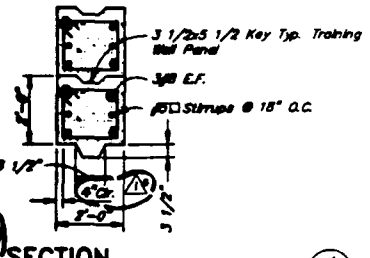
CAST-IN-PLACE CONCRETE TRAINING WALL

Scale: 1" = 3'



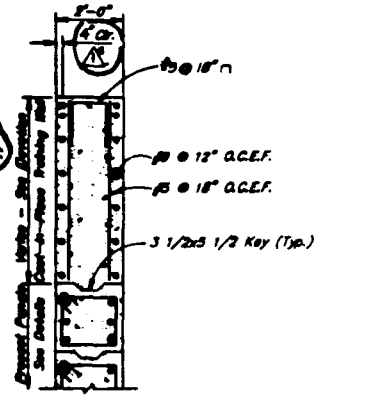
TRAINING WALL FOOTING DETAIL

N.T.S.



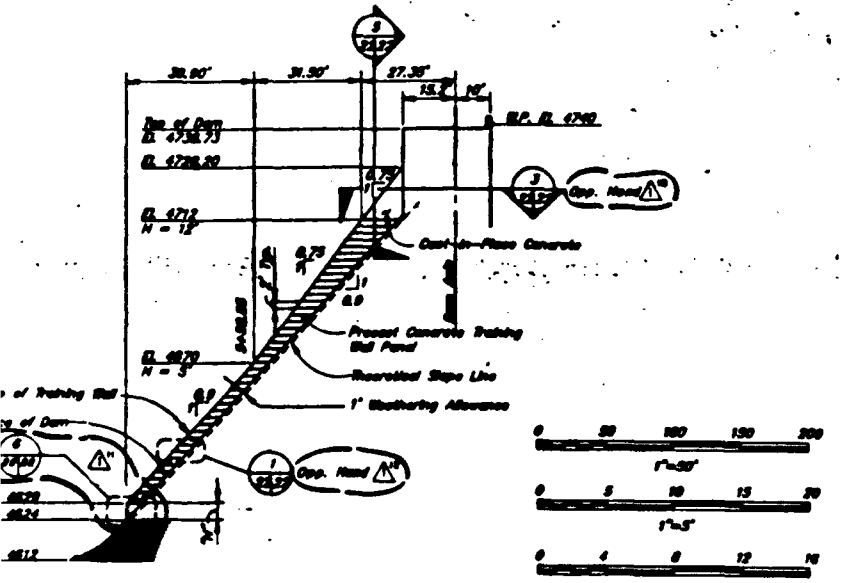
SECTION

Scale: 1" = 3'



SECTION

Scale: 1" = 3'



TRAINING WALL ELEVATION

Scale: 1" = 20'; Horiz & Vert

	Change Gr. to 5', Remove Rebar, Add Bench Sketch (Alt. #2)	Aug. 1989	
SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

BOYLE ENGINEERING CORPORATION CONSULTING ENGINEERS ALBUQUERQUE, NEW MEXICO	U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE CORPS OF ENGINEERS ALBUQUERQUE, NEW MEXICO

RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

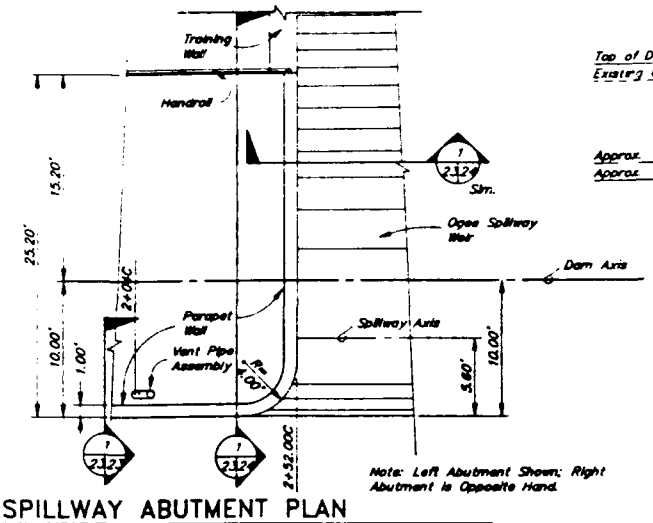
RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

DAM
STRUCTURAL PLAN, ELEVATIONS & SECTIONS

DESIGNED BY: REP/TWP
DRAWN BY: MEA
CHECKED BY: JFB

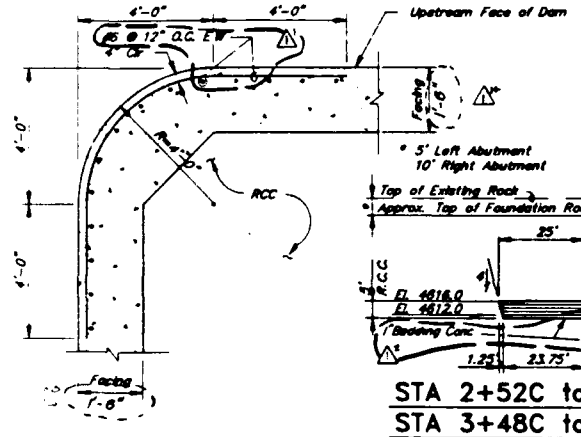
Fred Brown
ENGINEER
DATE JULY, 1989

SOL.NO.DACW47-89-B-0024 PLATE
FILE NUMBER RG-TC-A-22.1 A-29



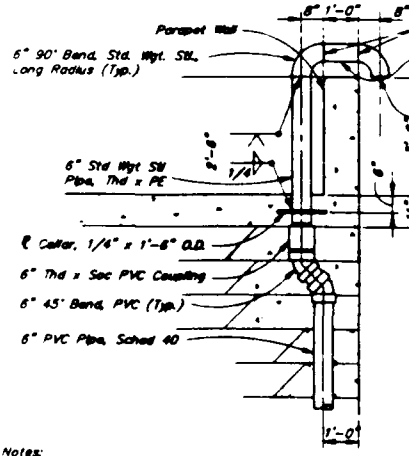
SPILLWAY ABUTMENT PLAN

Scale: 1"=5'



SPILLWAY CORNER DETAIL

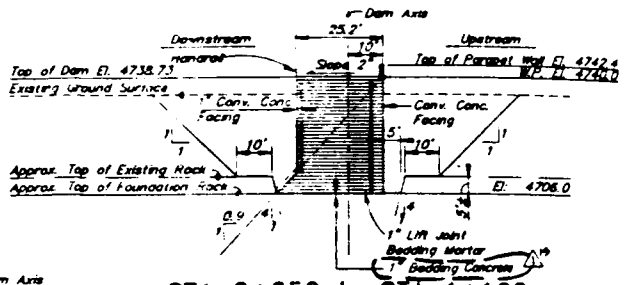
Scale: 1/2" = 1'-0"



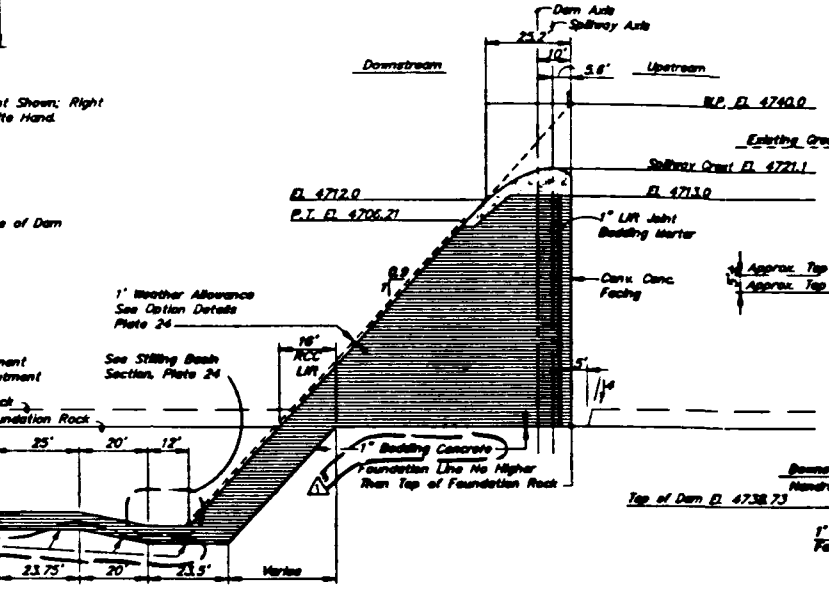
Notes:
1. Standard Weight Steel Pipe Shall Conform To The Specification For Steel Pipe Used For The Handrail.
2. Galvanized Entire Steel Vent Pipe Assembly After Fabrication.
3. Align 90° Bends Perpendicular To Face Of Parapet Wall.

VENT PIPE DETAIL

Scale: 1/2" = 1'-0"



STA 0+25C to STA 1+16C



STA 2+52C to STA 2+80C
STA 3+48C to STA 3+72C

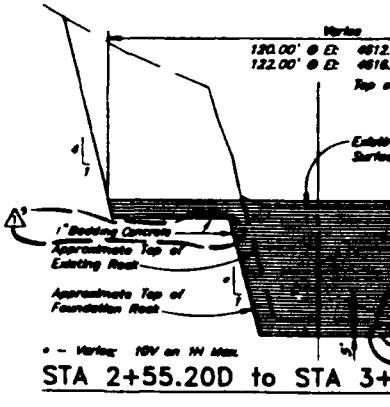
TYPICAL DAM SECTIONS

Scale: 1" = 20'

Scale: 1/2" = 1'-0"

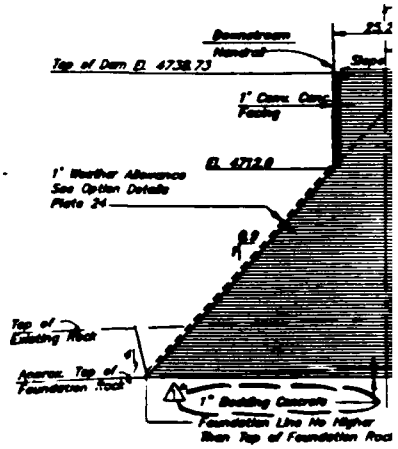
Scale: 1" = 5'

Scale: 1" = 20'

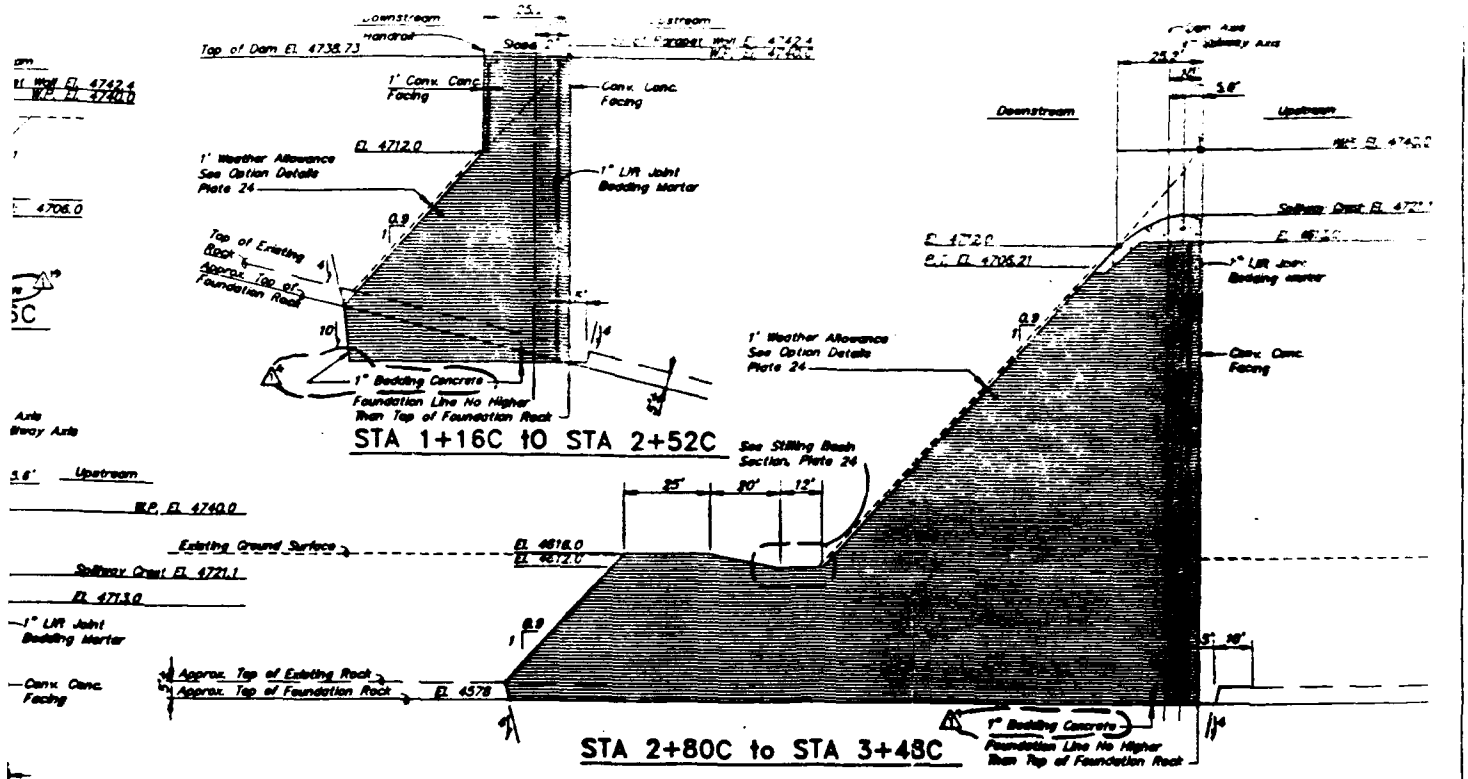


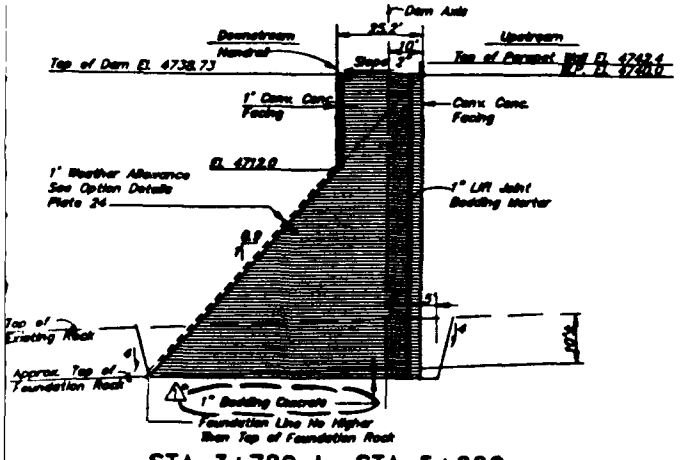
• - Varies: 10V on 1H Max.

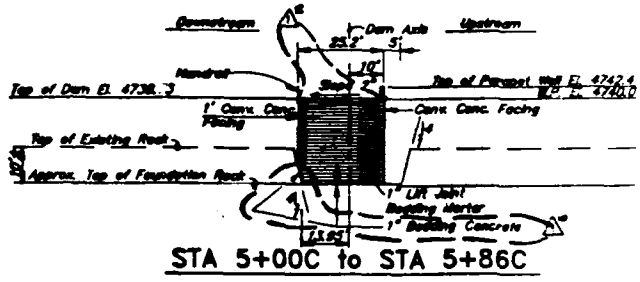
STA 2+55.20D to STA 3+12.20D

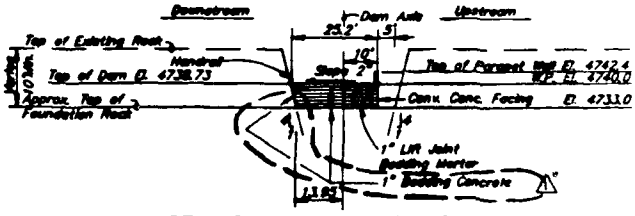


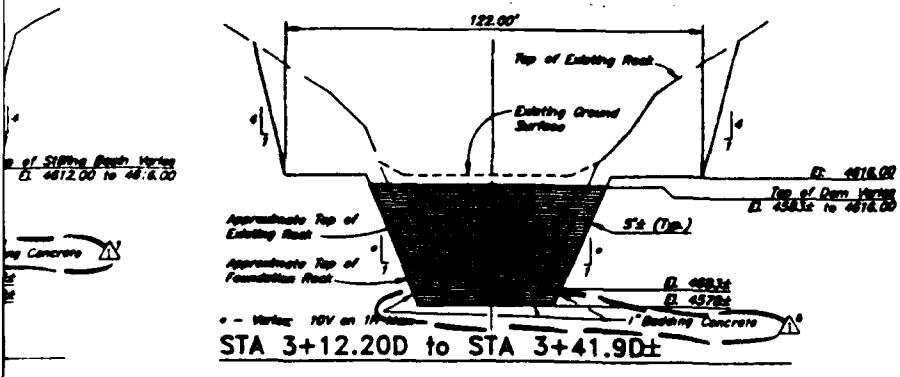
STA 3+72C to STA











1	CHANGE NOTATION ON SECTIONS (nm #2)	AUG 89	
SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

BOYLE ENGINEERING CORPORATION
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

DESIGNED BY RSP/TWP
DRAWN BY GEA
CHECKED BY FMB

RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

DAM
STRUCTURAL SECTIONS & DETAILS

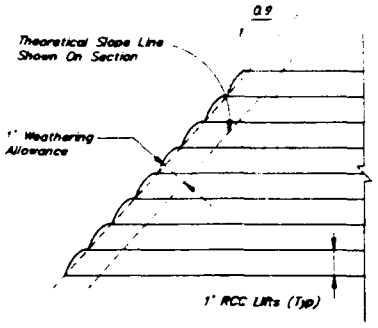
SOL. NO. DACW47-89-B-0024 PLATE

ENGINEER

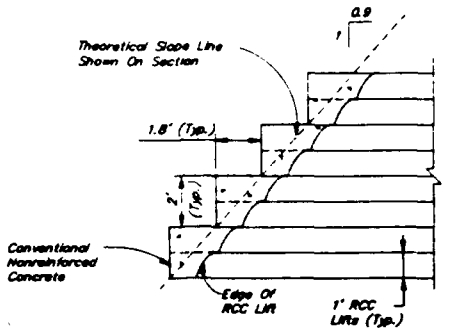
DATE JULY, 1989

FILE NUMBER RG-TC-A-23.1

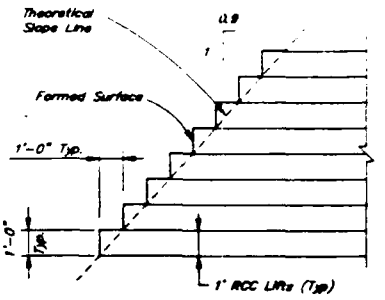
A-30



OPTION No. 1



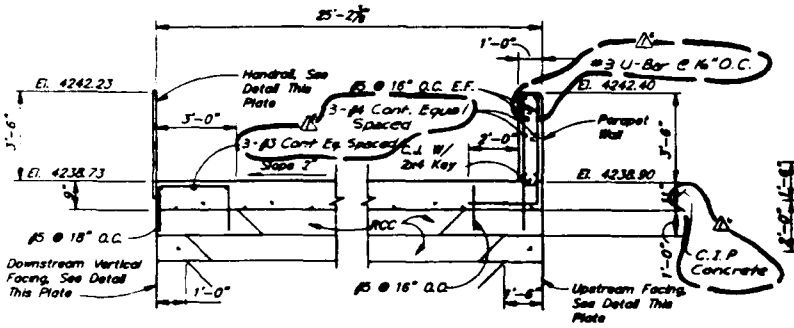
OPTION No. 2

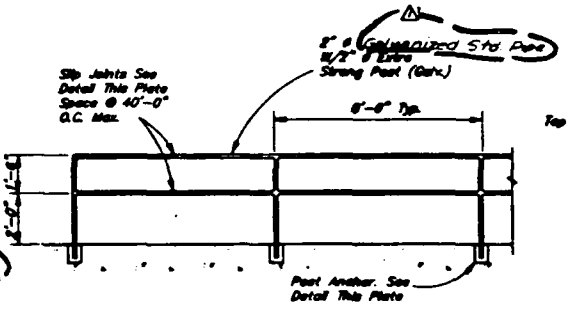


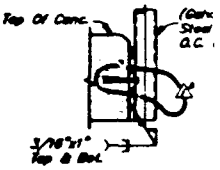
OPTION No. 3

DOWNSTREAM SLOPED FACE DETAIL

Scale: 3/8" = 1'-0"



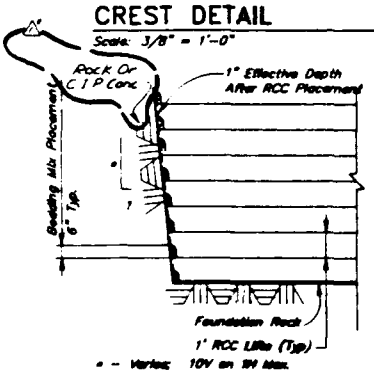




SIDE VIEW

CREST DETAIL

Scale: 3/8" = 1'-0"



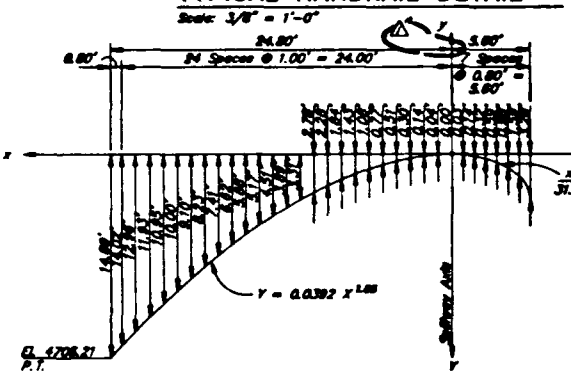
* - Varies: 10V on 9H Max.

BEDDING MIX PLACEMENT ON FDN. SLOPE DETAIL

Scale: 3/8" = 1'-0"

TYPICAL HANDRAIL DETAIL

Scale: 3/8" = 1'-0"

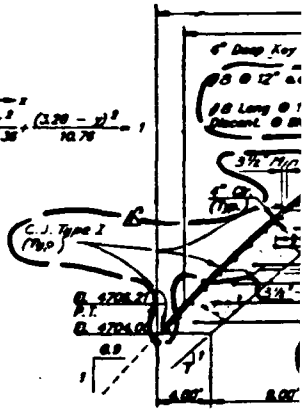


WEIR CURVE LAYOUT

Scale: 1" = 5'

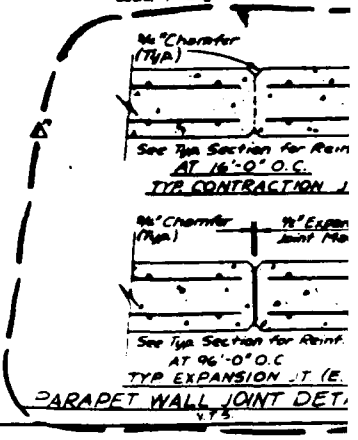
HANDRAIL

Scale: 1-1/2" = 1'

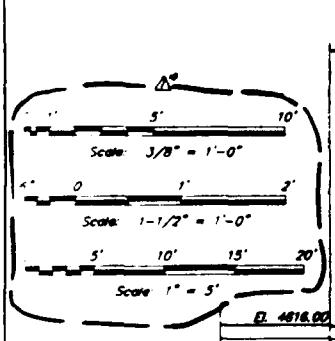


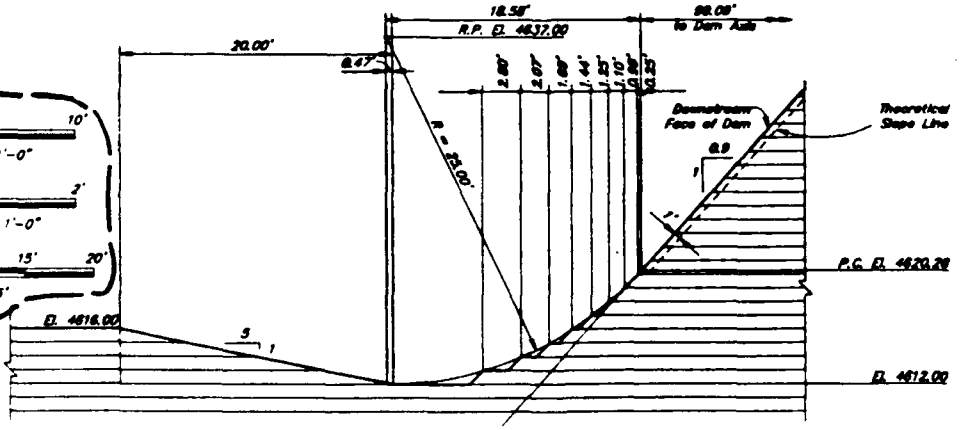
CONVENTIONAL C

Scale: 1" = 5'



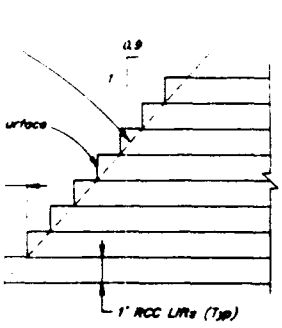
PARAPET WALL JOINT DET

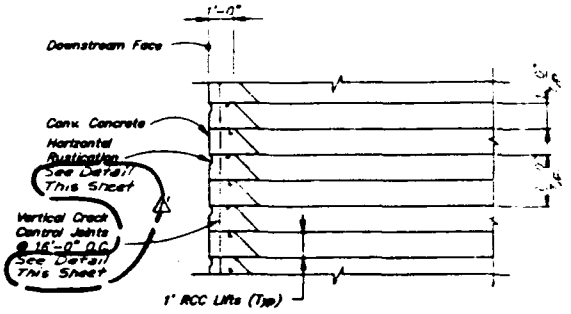


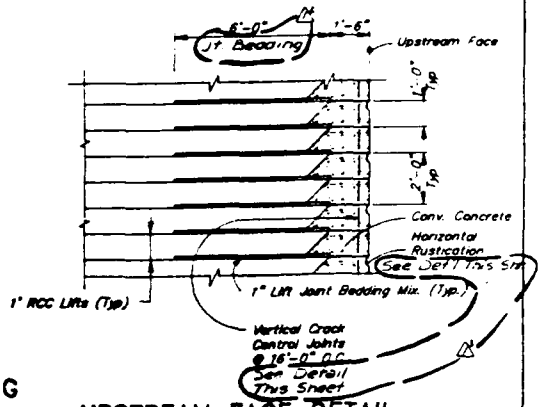


STILLING BASIN SECTION

Scale: 1" = 5'







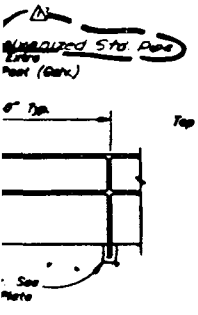
OPTION No. 3

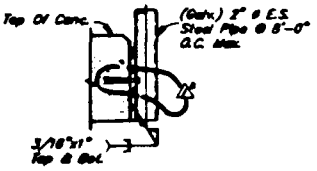
DOWNSTREAM VERTICAL FACING DETAIL ABOVE EL: 47[illegible].0

Scale: 3/8" = 1'-0"

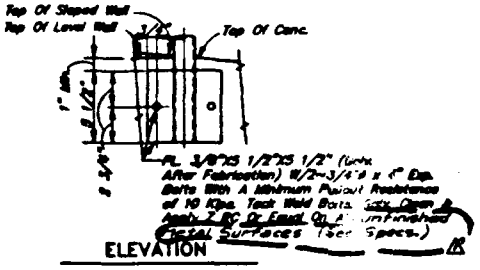
UPSTREAM FACE DETAIL

Scale: 3/8" = 1'-0"

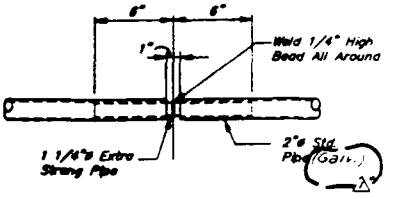




SIDE VIEW



ELEVATION



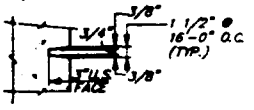
AIL

HANDRAIL POST ANCHOR DETAIL

Scale: 1-1/2" = 1'-0"

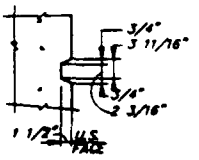
RAILING SLIP JOINT DETAIL

Scale: 1-1/2" = 1'-0"



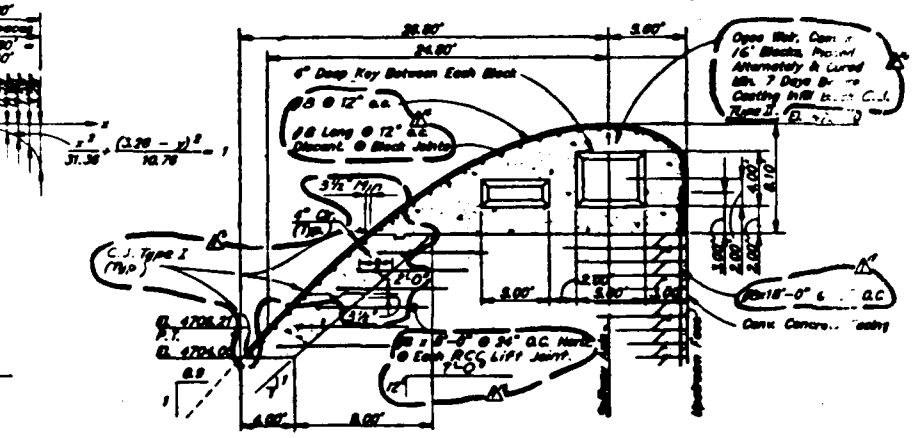
TYP. VERTICAL CRACK CONTROL JOINT DETAIL

Scale: 1-1/2" = 1'-0"



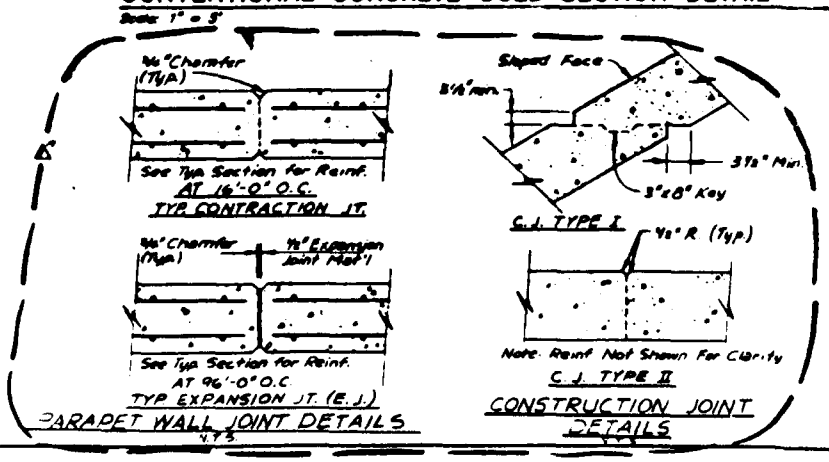
HORIZONTAL RUSTICATION DETAIL

Scale: 1-1/2" = 1'-0"



CONVENTIONAL CONCRETE OGEE SECTION DETAIL

Scale: 1" = 5'



MK	DESCRIPTIONS	DATE	APPROVAL
	Sht 2. As Per Amendment No. 2	Aug 89	

REVISIONS

BOYLE ENGINEERING CORPORATION
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

DESIGNED BY: WM/FB
DRAWN BY: CEA
CHECKED BY: JFB

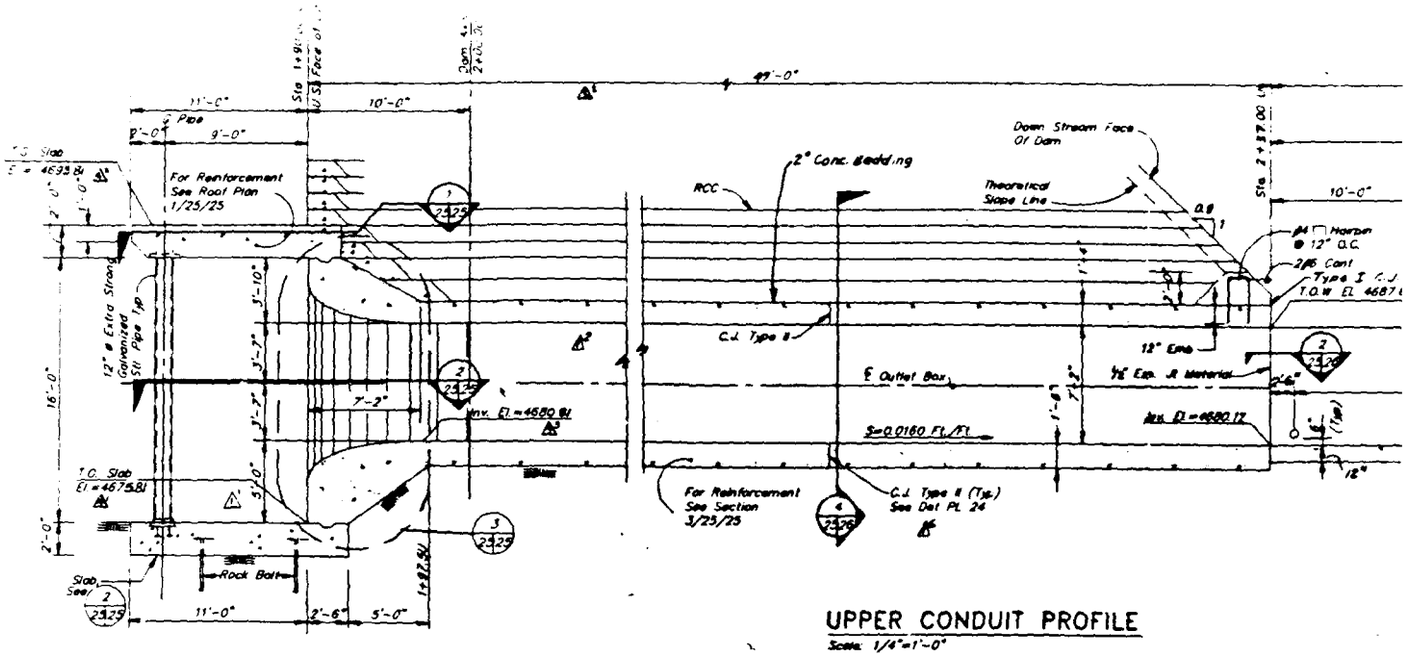
RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

DAM
STRUCTURAL SECTION & DETAILS

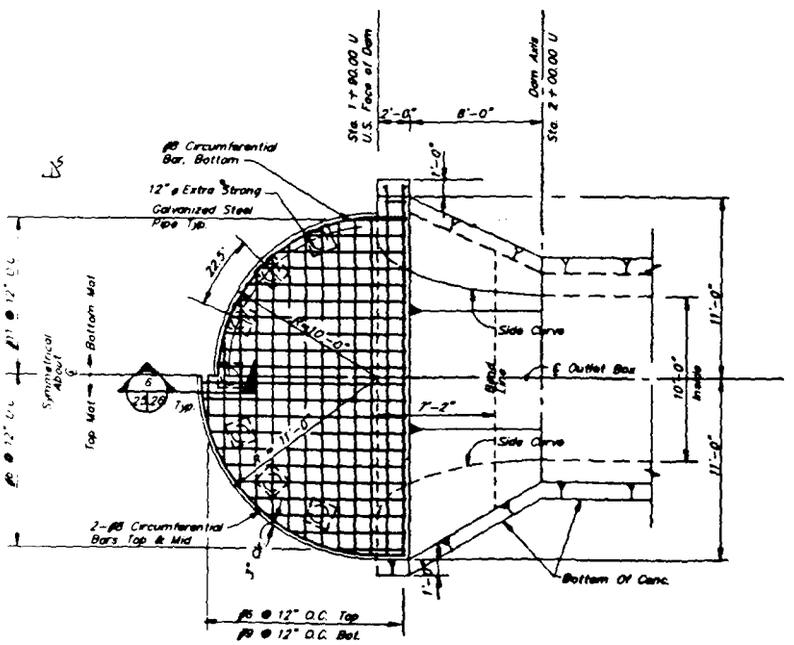
ENGINEER
DATE JULY, 1989

SOL.NO.DACW47-89-B-0024 PLATE
FILE NUMBER RG-TC-A-24. A-31



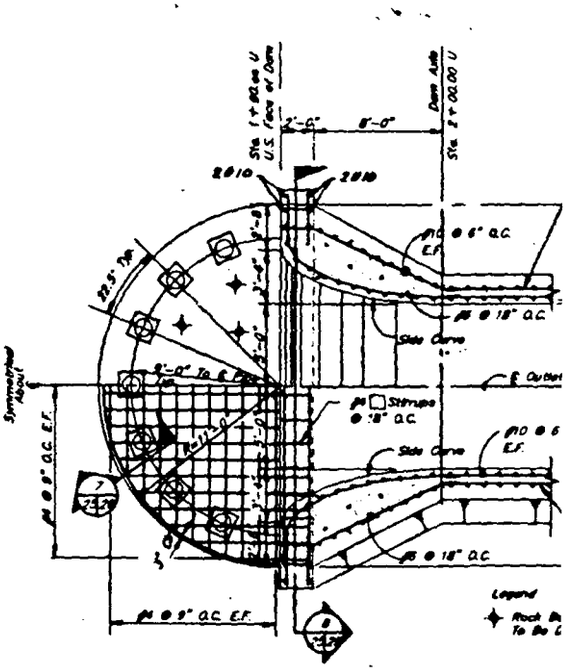
UPPER CONDUIT PROFILE

Scale: 1/4"=1'-0"



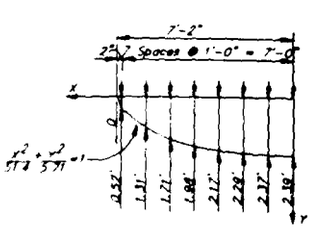
ENTRANCE ROOF PLAN

Scale: 1/4"=1'-0"



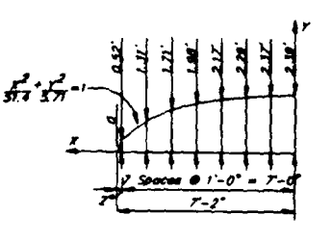
ENTRANCE FOUNDATION

Scale: 1/8"=1'-0"



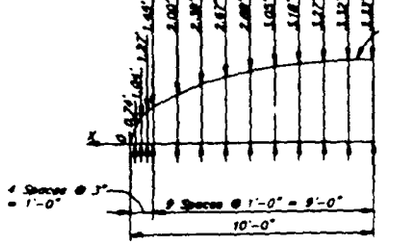
ROOF CURVE LAYOUT

Scale: 3/8"=1'-0"



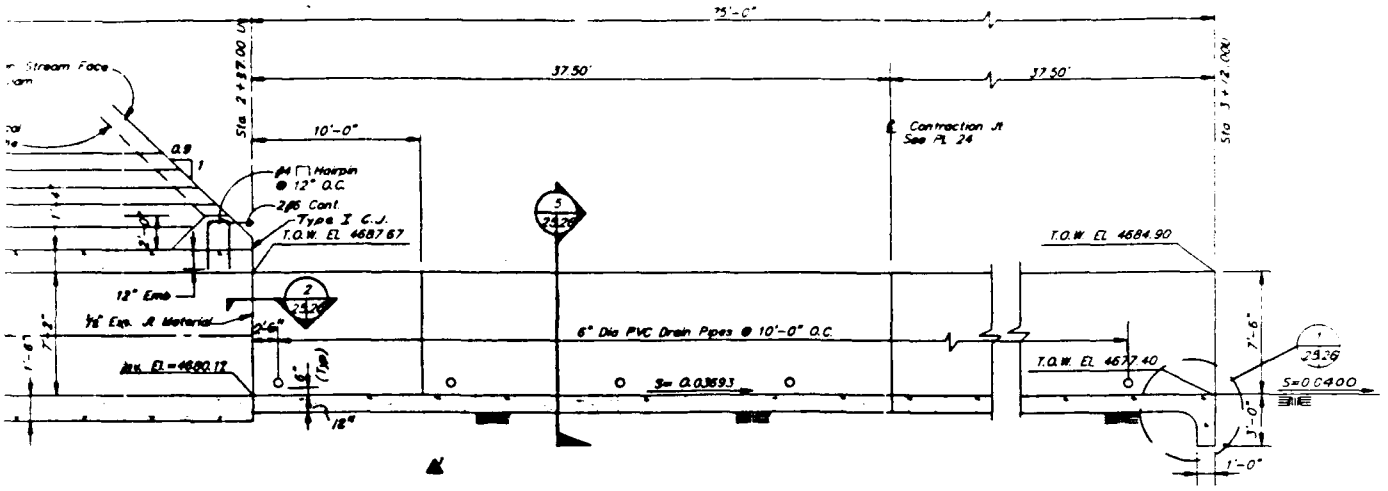
BOTTOM CURVE LAYOUT

Scale: 3/8"=1'-0"

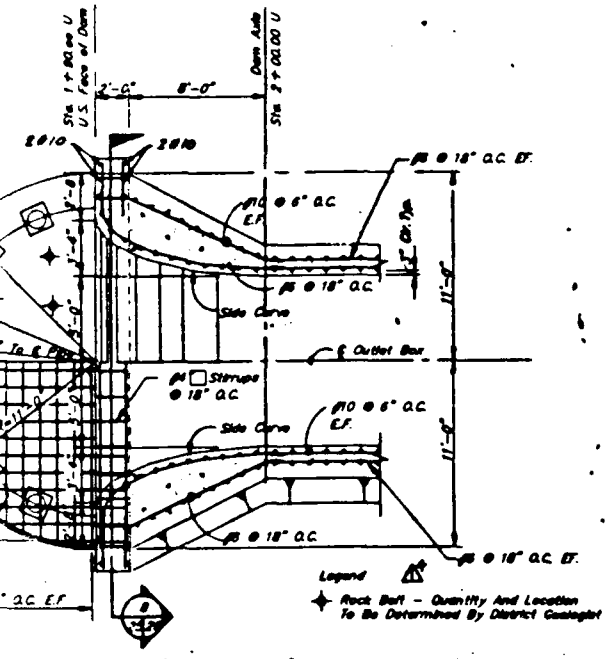


SIDE CURVE LAYOUT

Scale: 3/8"=1'-0"

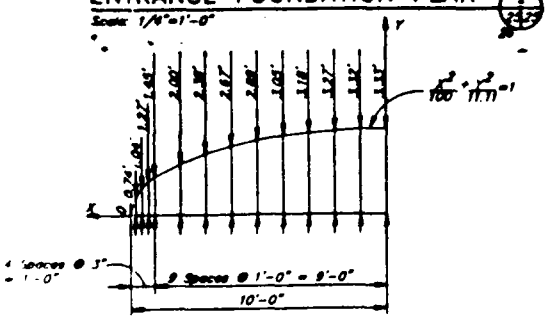


PROFILE



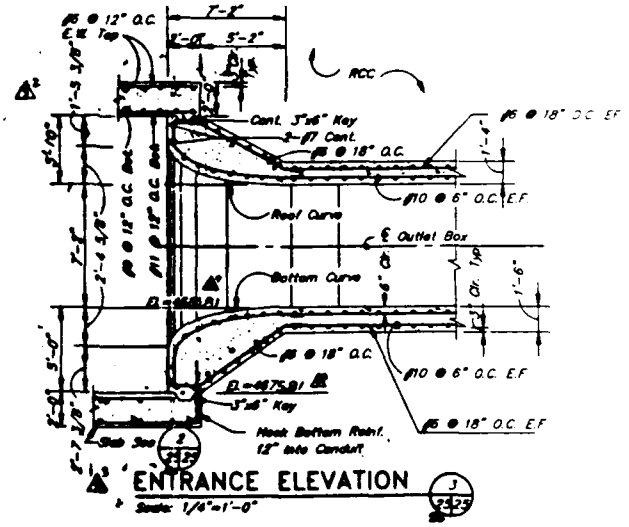
ENTRANCE FOUNDATION PLAN

Scale 1/8"=1'-0"



SIDE CURVE LAYOUT

Scale 3/8"=1'-0"



ENTRANCE ELEVATION

Scale: 1/8"=1'-0"

	"AS BUILT"		
	REV. D.3" [illegible]		7/89/91
	Rev. Prelims. 1/86/88, 3/30/88 & 3/85/89		MAY 90
	Revised Upper Gallery Roofing (Bl. 24) T		APR 90
	Revised Lower Gallery Roof Conc. Detailing (Alt B.2)		AUG 1989
SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

BOYLE ENGINEERING CORPORATION CONSULTING ENGINEERS ALBUQUERQUE, NEW MEXICO	U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE CORPS OF ENGINEERS ALBUQUERQUE, NEW MEXICO

DESIGNED BY RSP/JFB
DRAWN BY RJA
CHECKED BY FO/JFB

RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

HIGH LEVEL OUTLET WORKS

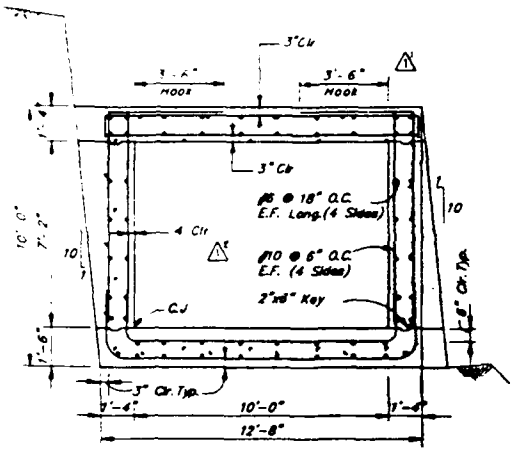
STRUCTURAL PLANS, PROFILE & ELEVATION

Fred Brown
ENGINEER
DATE JULY, 1989

SOL. NO. DACW47-89-B-0024 PLATE

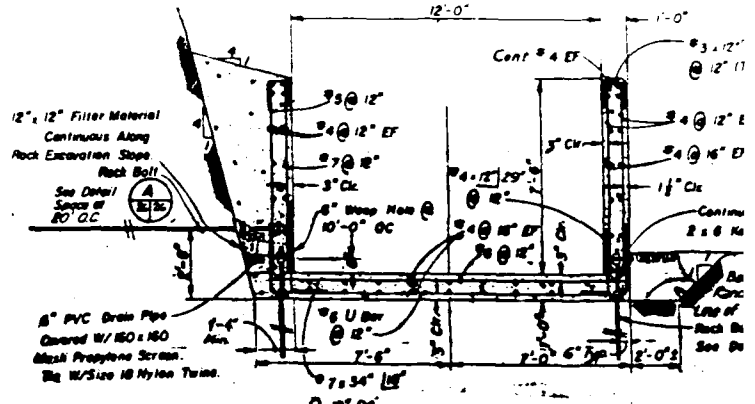
FILE NUMBER RG-TC-A- 25.4 A-32

A-33 ML 8479



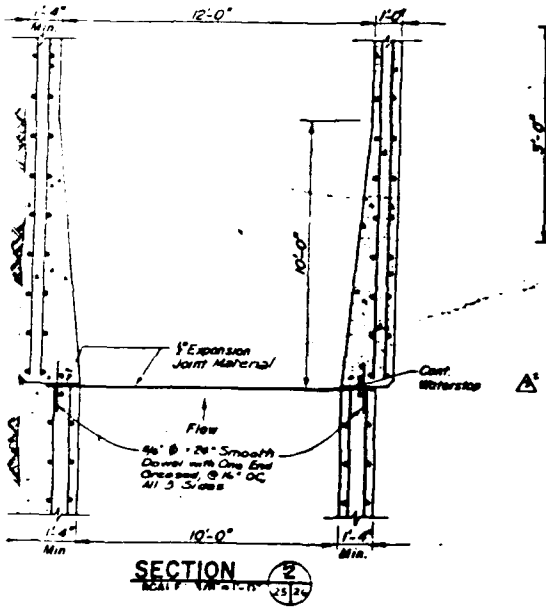
SECTION STA. 1+97.50U TO 2+37U

Scale 3/8"=1'-0"

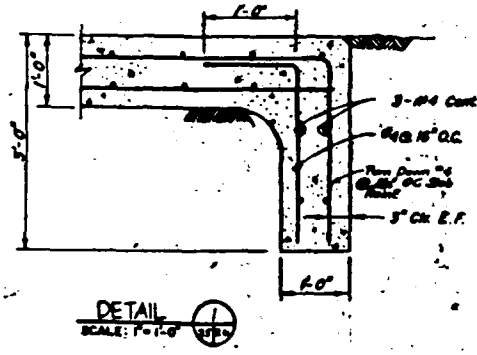


SECTION STA. 2+37U TO 3+12U

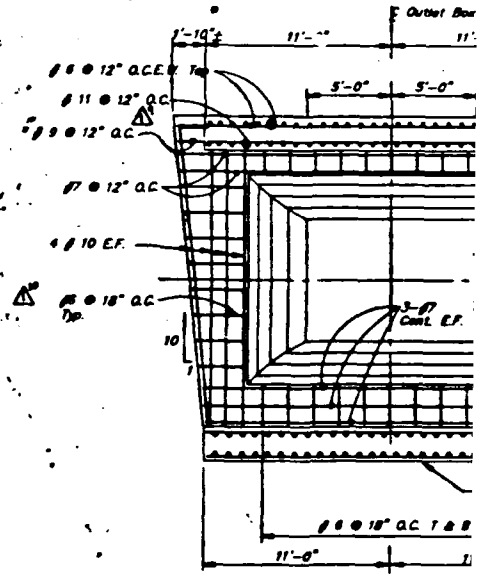
Scale 3/8"=1'-0"



SECTION

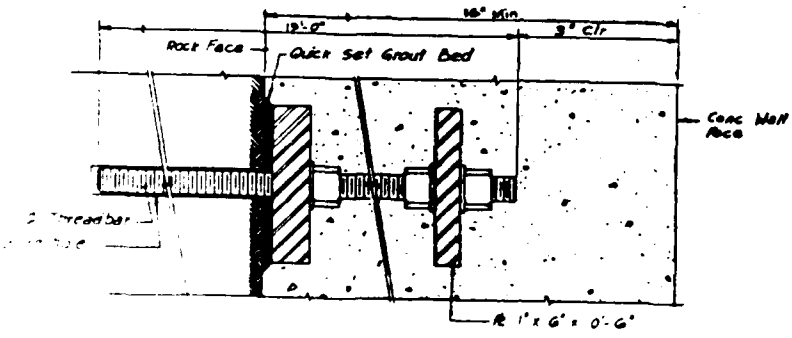


DETAIL



ENTRANCE ELEVATION

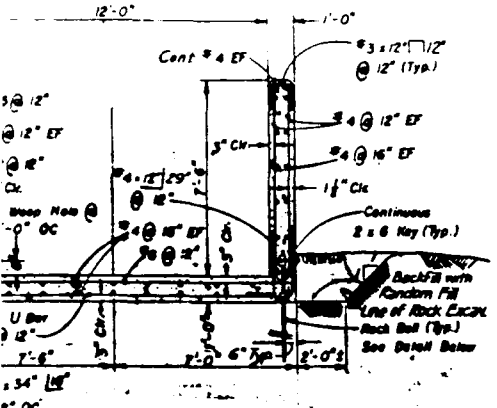
Scale 1/4"=1'-0"



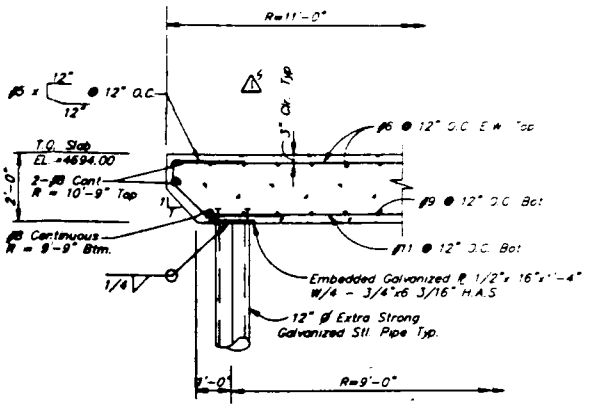
ROCK BOLT DETAIL

CONCRETE STRENGTH TABLE

Item	Psi 28 Day Min. Compressive Strength
Dam & Spillway Chute	3000 Psi.
Upstream & Downstream Dam Facing	3000 Psi.
Intake Sluice	3000 Psi.
Concrete Encasement	3000 Psi.
Dam Non-Overflow Core	3000 Psi.
Support Backfill For RCP	3000 Psi.
Existing Concrete For Present Training Walls	3000 Psi.
Dam Parapet Wall	4000 Psi.
High Level Outlet Inlet	4000 Psi.
Intake Tower (Cast in Place & Precast)	4000 Psi.
Training Walls (Cast in Place & Precast)	4000 Psi.

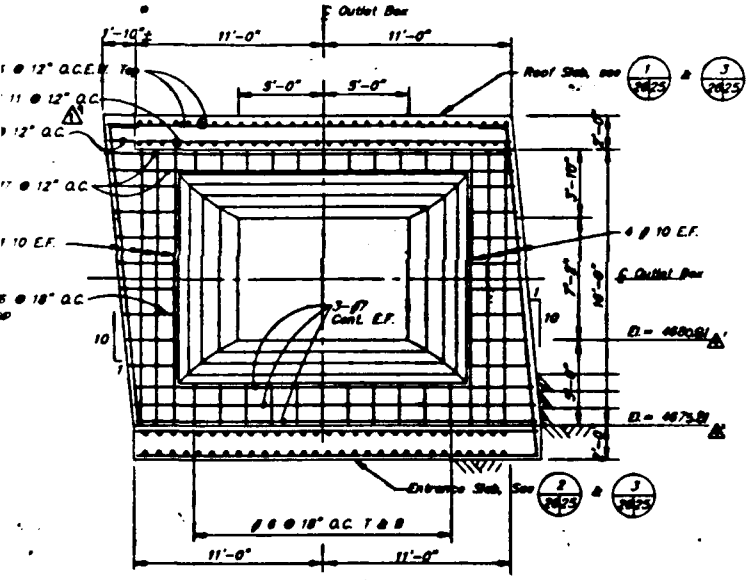


STA. 2+37U TO 3+12U



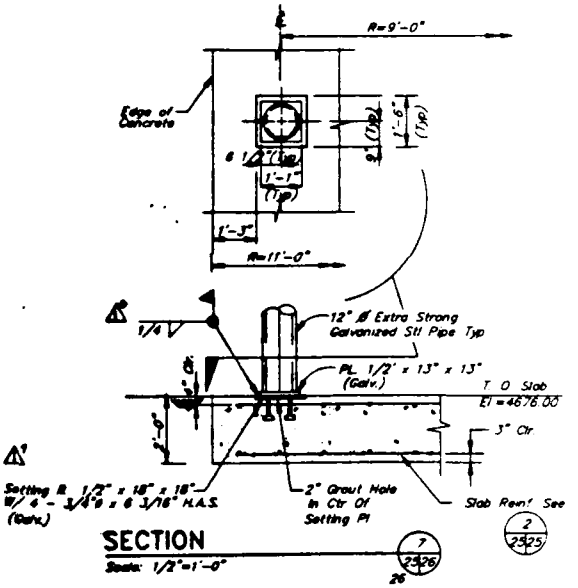
SECTION

Scale: 1/2"=1'-0"



ENTRANCE ELEVATION

Scale: 1/4"=1'-0"



SECTION

Scale: 1/2"=1'-0"

NCRETE STRENGTH TABLE

	Pc 28 Day Min. Compressive Strength
	3000 Psi
ure Fighting	3000 Psi
	3000 Psi
	3000 Psi
	3000 Psi
ent Training Walls	3000 Psi
	4000 Psi
	4000 Psi
& Precast)	4000 Psi
& Precast)	4000 Psi

	ADD EL. 4680.81, 4673.81 "AS BUILT"	7/29/91	
	ADD SLAB @ ELEV. 3/32/36, 3/36/36 & SAS. (ML 8479)	MAY 90	
	ADD ROCK BOLT DETAIL (ML 8447)	APR 90	
	REVISED SECTION 5; ADD DETAIL 1; ADD SECTION 2;		
	ADD TABLE (AR 82)	AUG 89	
	CHANGE NOTES, ADD/CHANGE REBAR, PL CNT		
TRIGRAL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

BOYLE ENGINEERING CORPORATION CONSULTING ENGINEERS ALBUQUERQUE, NEW MEXICO	U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE CORPS OF ENGINEERS ALBUQUERQUE, NEW MEXICO

DESIGNED BY: RSP/FB
DRAWN BY: BJH
CHECKED BY: FD/FB

RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

HIGH LEVEL OUTLET WORKS

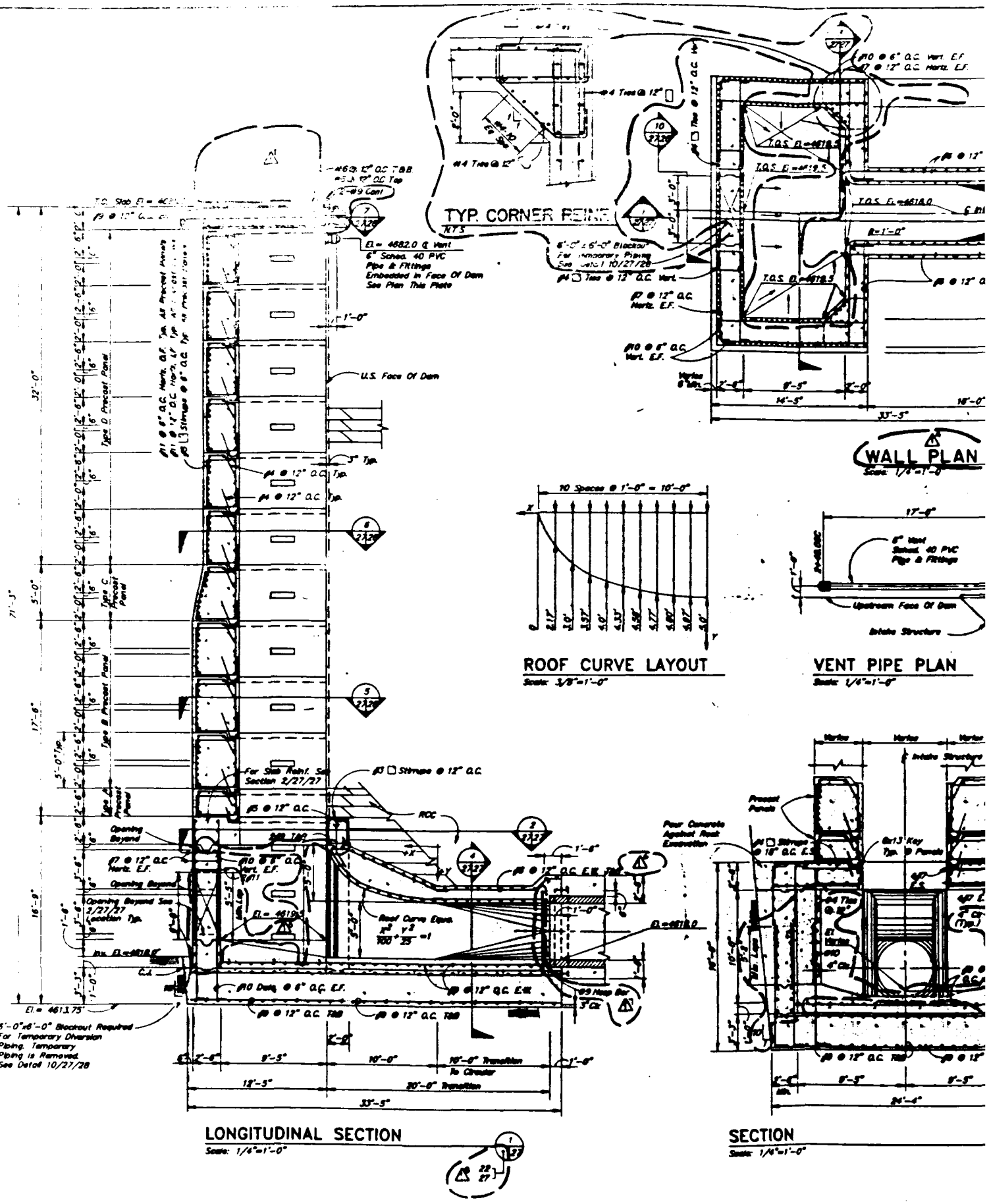
STRUCTURAL SECTIONS & ELEVATION

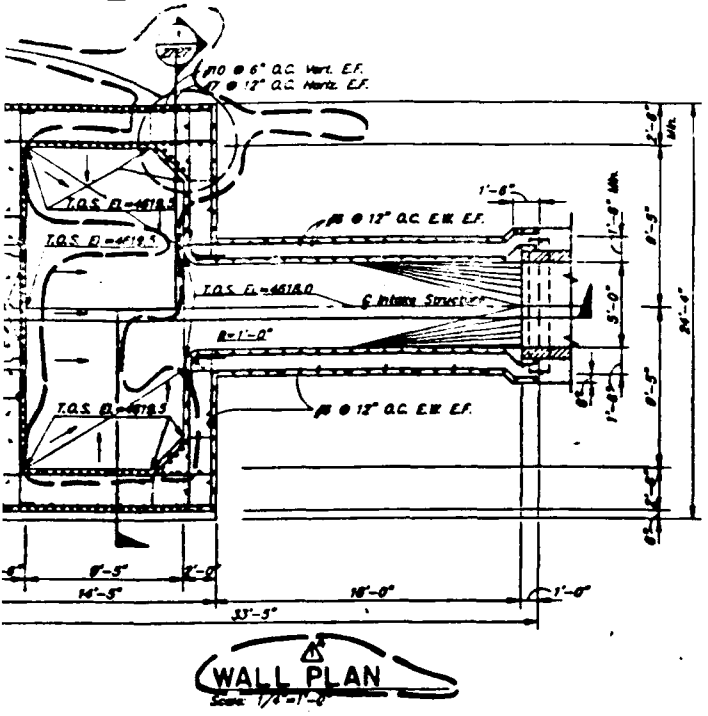
Fred Burns
ENGINEER
DATE JULY, 1989

SOL. NO. DACW47-89-B-0024 PLATE

FILE NUMBER RG-TC-A-26.4 A-33

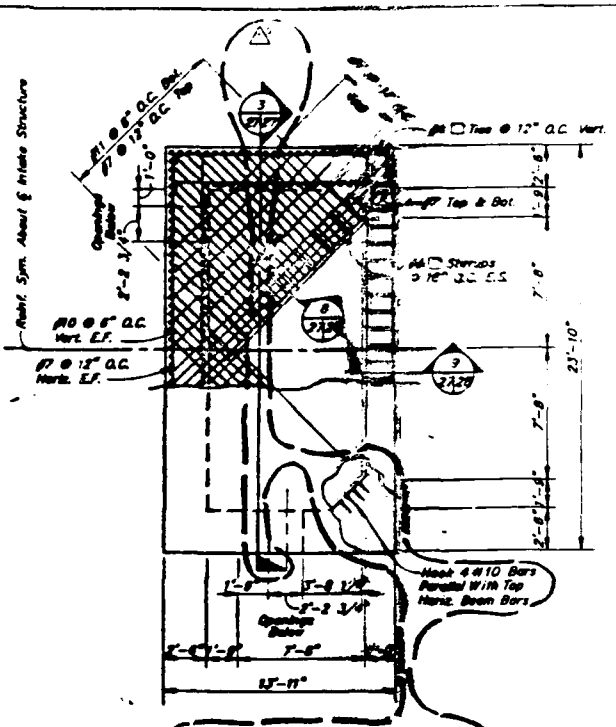
DRAWING OF WORK "AS BUILT" ML 8479 A-34


TYP. CORNER REINF.
NTS
WALL PLAN
Scale: 1/4"=1'-0"
ROOF CURVE LAYOUT
Scale: 3/8"=1'-0"
VENT PIPE PLAN
Scale: 1/4"=1'-0"
LONGITUDINAL SECTION
Scale: 1/4"=1'-0"
SECTION
Scale: 1/4"=1'-0"
U.S. Face Of Dam
Upstream Face Of Dam
Intake Structure
Roof Curve Eqn.
6'-0"x6'-0" Blockout Required For Temporary Diversion Piping. Temporary Piping Is Removed. See Detail 10/27/28
10'-0" Transition
30'-0" Transition
To Chester
33'-5"



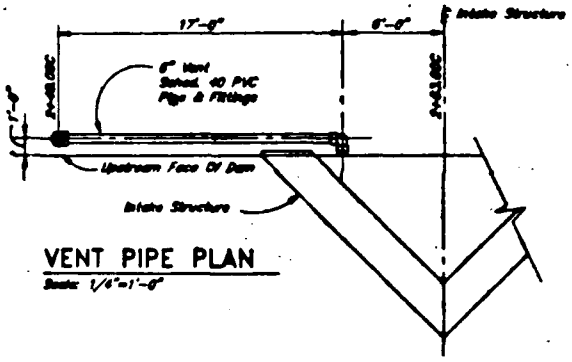
WALL PLAN

Scale: 1/4"=1'-0"



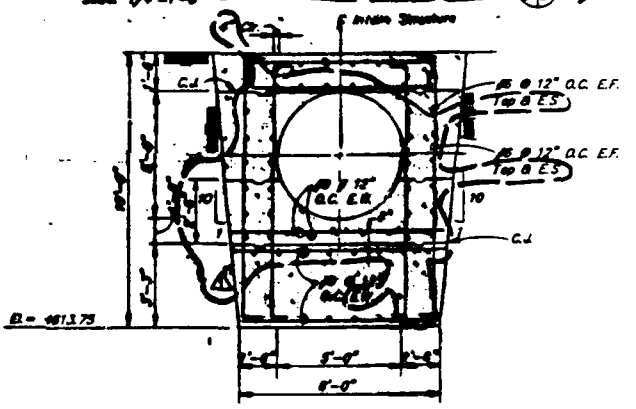
SECTION (ROOF SLAB PLAN VIEW) 2/2.27

Scale: 1/4"=1'-0"



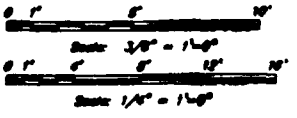
VENT PIPE PLAN

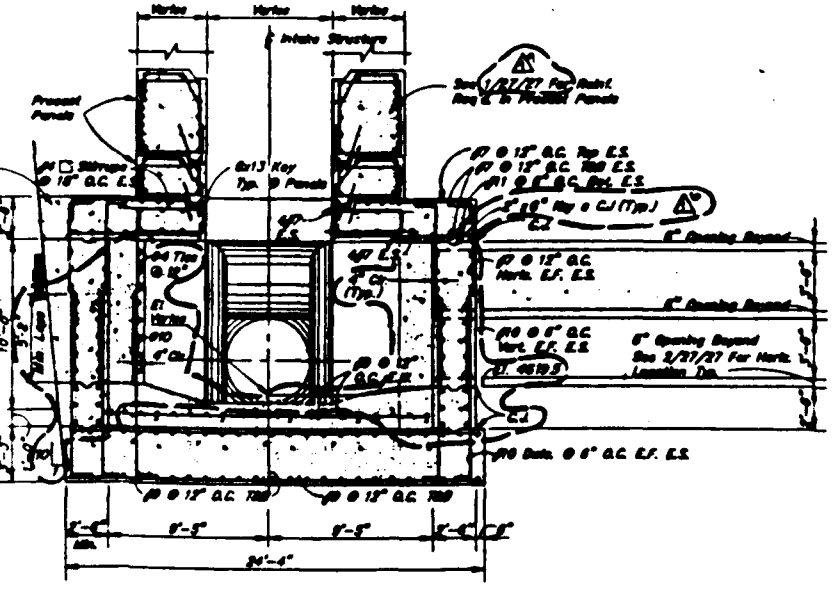
Scale: 1/4"=1'-0"



SECTION 4/2.27

Scale: 3/8"=1'-0"





SECTION

Scale: 1/4"=1'-0" 3/2.27

	REVISED SECTION [illegible] BEAM #2	AUG '89	
SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

BOYLE ENGINEERING CORPORATION CONSULTING ENGINEERS ALBUQUERQUE, NEW MEXICO	U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE CORPS OF ENGINEERS ALBUQUERQUE, NEW MEXICO

DESIGNED BY: RBP/JFB
DRAWN BY: RJM
CHECKED BY: FD/JFB

RIO GRANDE & TRIBUTARIES — TRUTH OR CONSEQUENCES, N.M.

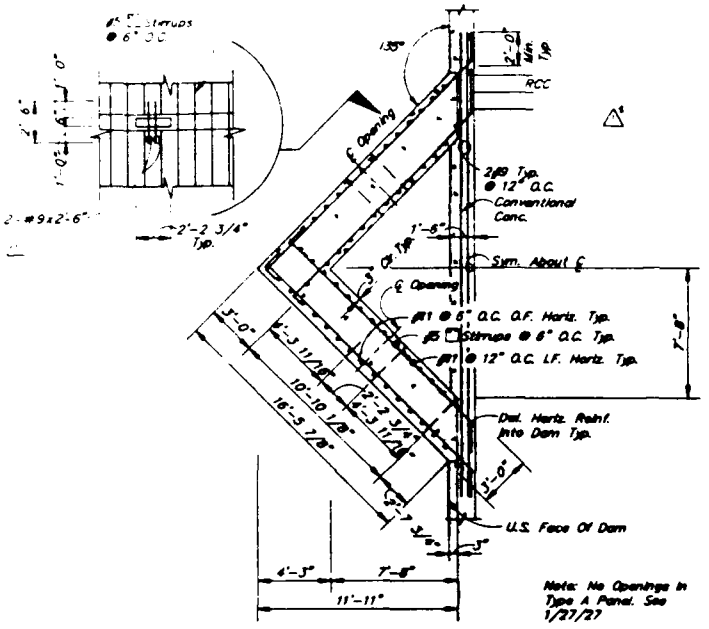
RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

LOW LEVEL OUTLET WORKS
INTAKE STRUCTURE
STRUCTURAL PLANS & SECTIONS

ENGINEER
DATE JULY, 1989

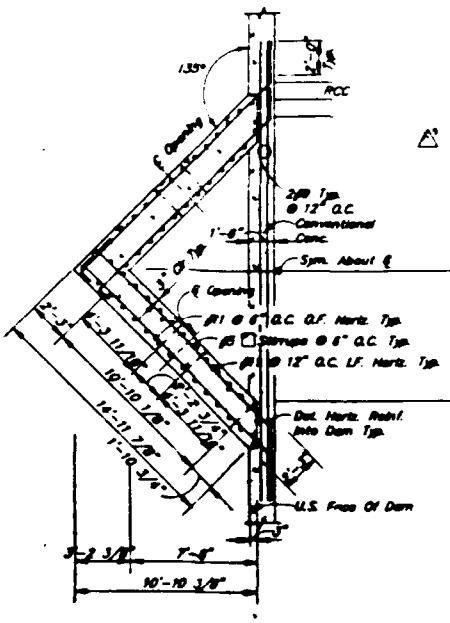
CONTRACT NO. DACW47-89-B-0024 — PLATE

FILE NUMBER RG-TC-A-27.1 — A-34



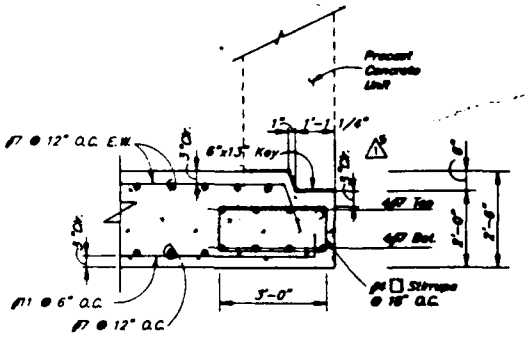
PRECAST PANEL DETAIL–TYPE A,B,& C Base

Scale: 1/4"=1'-0"



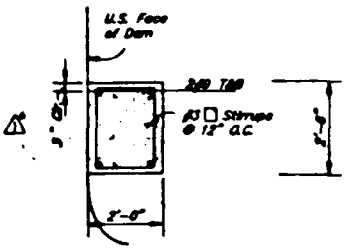
PRECAST PANEL DETAIL–TYPE C Top & D

Scale: 1/4"=1'-0"



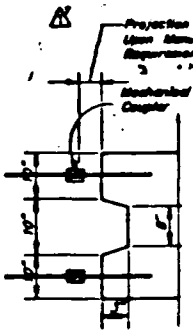
SECTION

N.T.S.



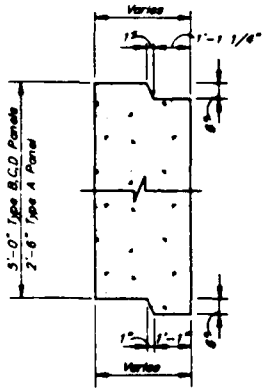
SECTION

N.T.S.



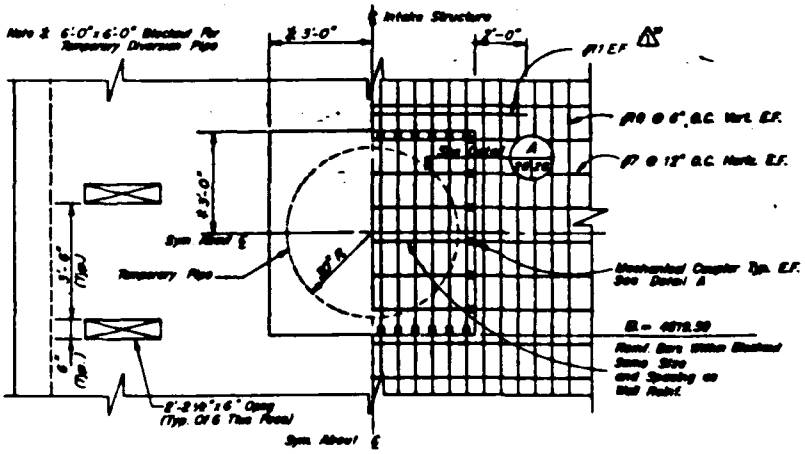
DETAIL A

N.T.S.



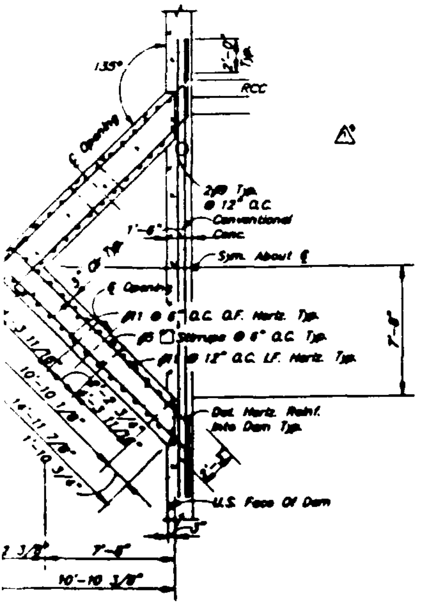
TYPICAL PRECAST PANEL SECTION

N.T.S.

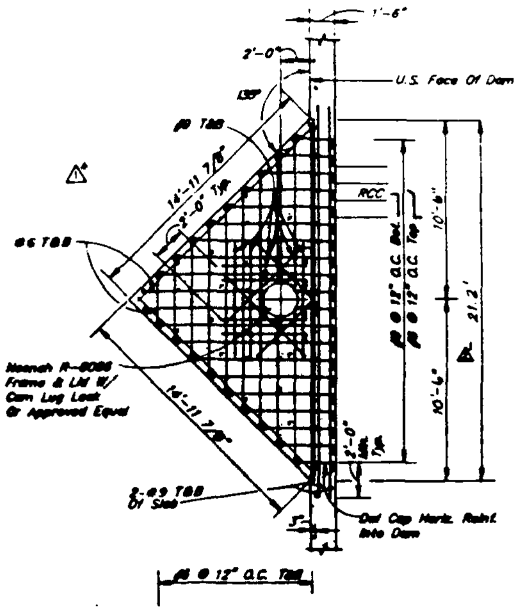


TEMPORARY BLOCKOUT ELEVATION

N.T.S.

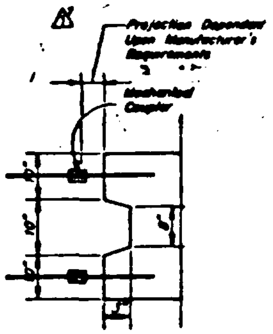


PANEL DETAIL–TYPE C & D



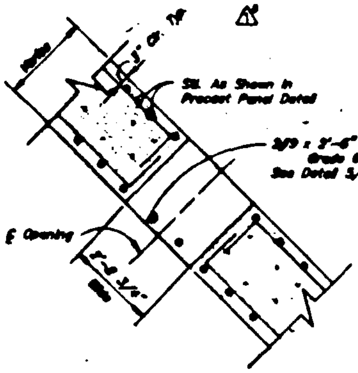
PRECAST CAP DETAIL

Scale: 1/4"=1'-0"



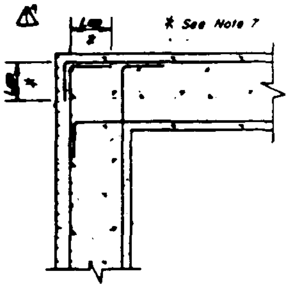
DETAIL A

N.T.S.



TYPICAL REINF. @ TOWER INLET

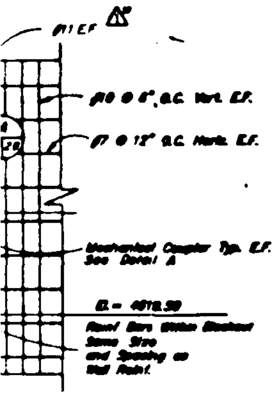
N.T.S.



INTAKE STRUCTURE CORNER DETAIL (TYP.)

N.T.S.

Scale: 1/4" = 1'-0"



GENERAL STRUCTURAL NOTES:

1. Low Level Outlet Works Intake Structure May Be Cast In Place @ The Contractors Option. Reinforcement Details Must Have Prior Approval Of The Contracting Officer
2. All Exposed Concrete Formed Surfaces Shall Be Class "B" Finish Except As Otherwise Noted. Class "B" Finish Shall Extend 1'-6" Below Finish Grade Or Backfill. Lower Unexposed Formed Surfaces Shall Be Class "D" Finish.
3. Interior Formed Surfaces Of Concrete Structures Shall Be Class "C" Finish, Except As Otherwise Noted On The Drawings.
4. A Float Finish Is Required For All Unformed Surfaces, Except As Noted On The Drawings.
5. Any Change In Construction Joints Must Have Prior Approval Of The Contracting Officer.
6. Chamfer All Exposed Concrete Edges & Construction Joints 3/4" Except As Noted On The Drawings.
7. Unless Otherwise Shown, Splices In Reinforcement Shall Be Class "C" Tension Splice Conforming To Reinforcement Tables 20.1 Thru 20.5 Of A.C.I. Publication SP–17(73).
8. Minimum Clear Cover For Reinforcement Shall Be 3" Except As Otherwise Noted.
9. Abbreviation C.J. Means Construction Joint.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REV PRECAST CAP DETAIL "AS BUILT"	7/29/91	
	CHANGE NOTES, DETAILS, & ELEVATION (AMD 2)		

REVISIONS

BOYLE ENGINEERING CORPORATION
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

DESIGNED BY: RSP/JFB
DRAWN BY: AM
CHECKED BY: FG/JFB

RIO GRANDE & TRIBUTARIES — TRUTH OR CONSEQUENCES, N.M.

RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

LOW LEVEL OUTLET WORKS INTAKE STRUCTURE

STRUCTURAL SECTIONS & DETAILS

SOL.NO.DACW47-89-B-0024

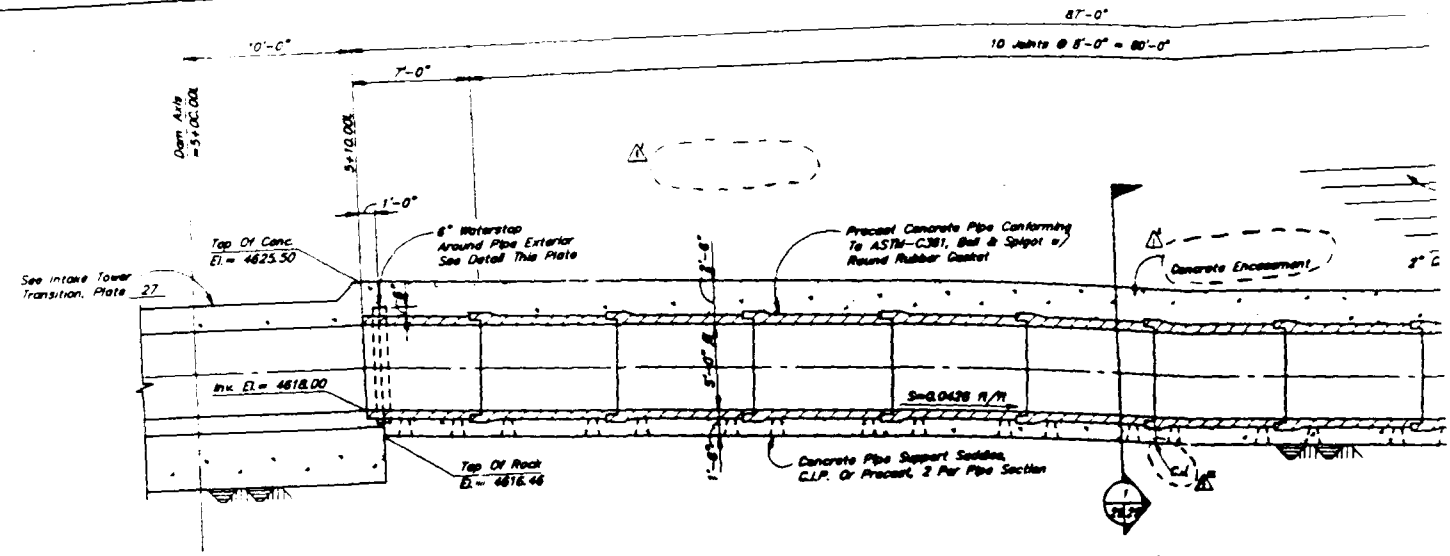
ENGINEER
DATE JULY, 1989

FILE NUMBER: RG–TC–A–28.

PLATE: A-35

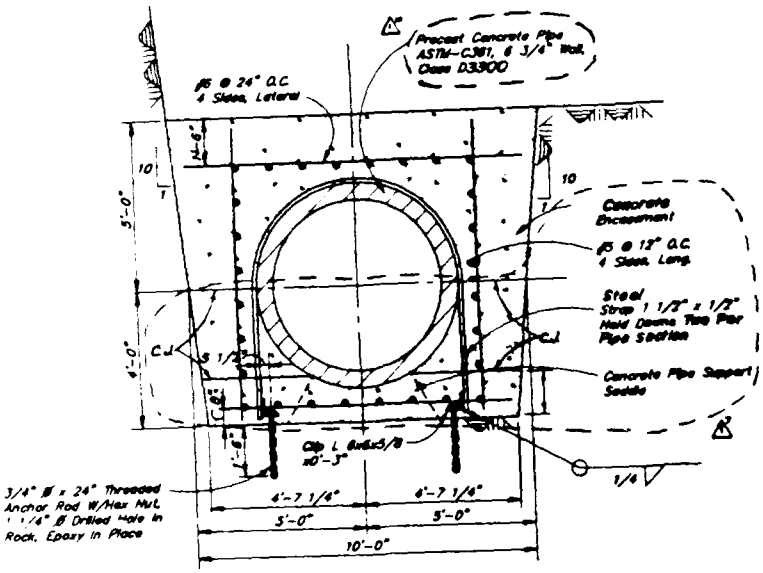
DRAWING OF WORK "AS BUILT" A-36

VALUE ENGINEERING PROPOSALS MEAN HIGHER PROFITS



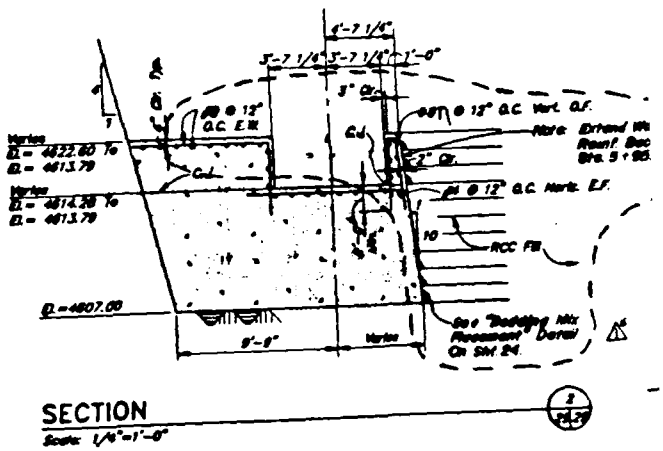
CONDUIT PROFILE

Scale: 1/4"=1'-0"



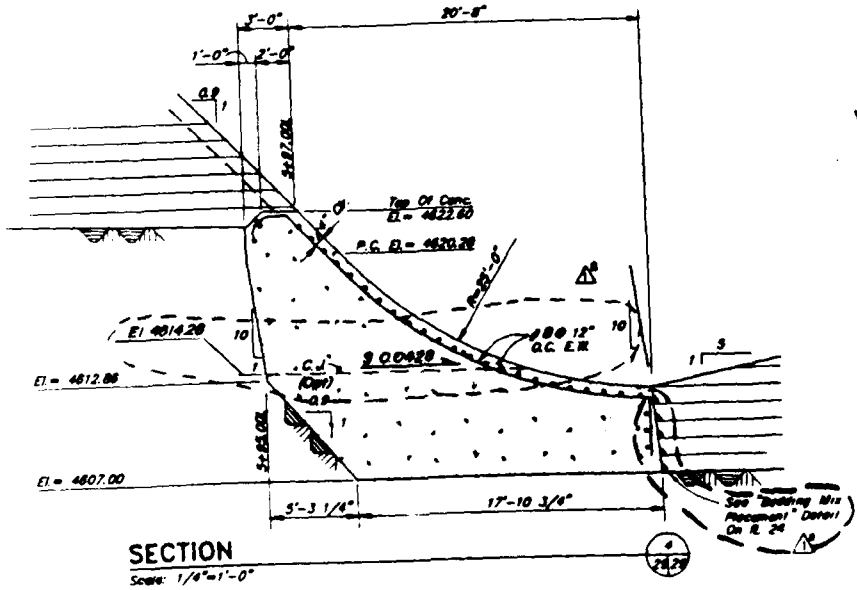
SECTION

Scale: 1/2"=1'-0"



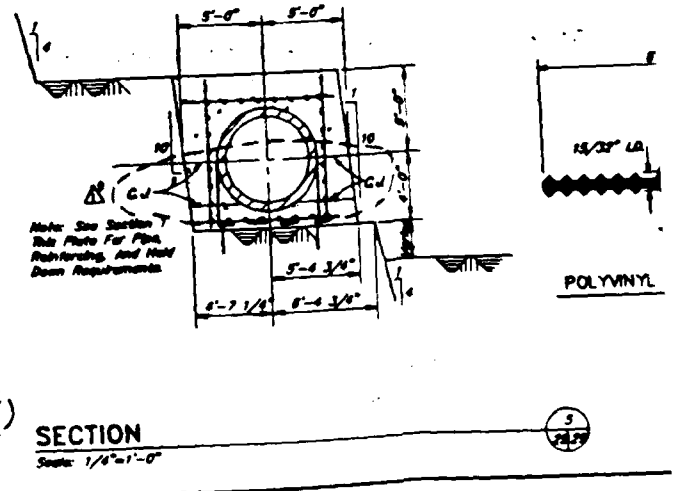
SECTION

Scale: 1/4"=1'-0"



SECTION

Scale: 1/4"=1'-0"

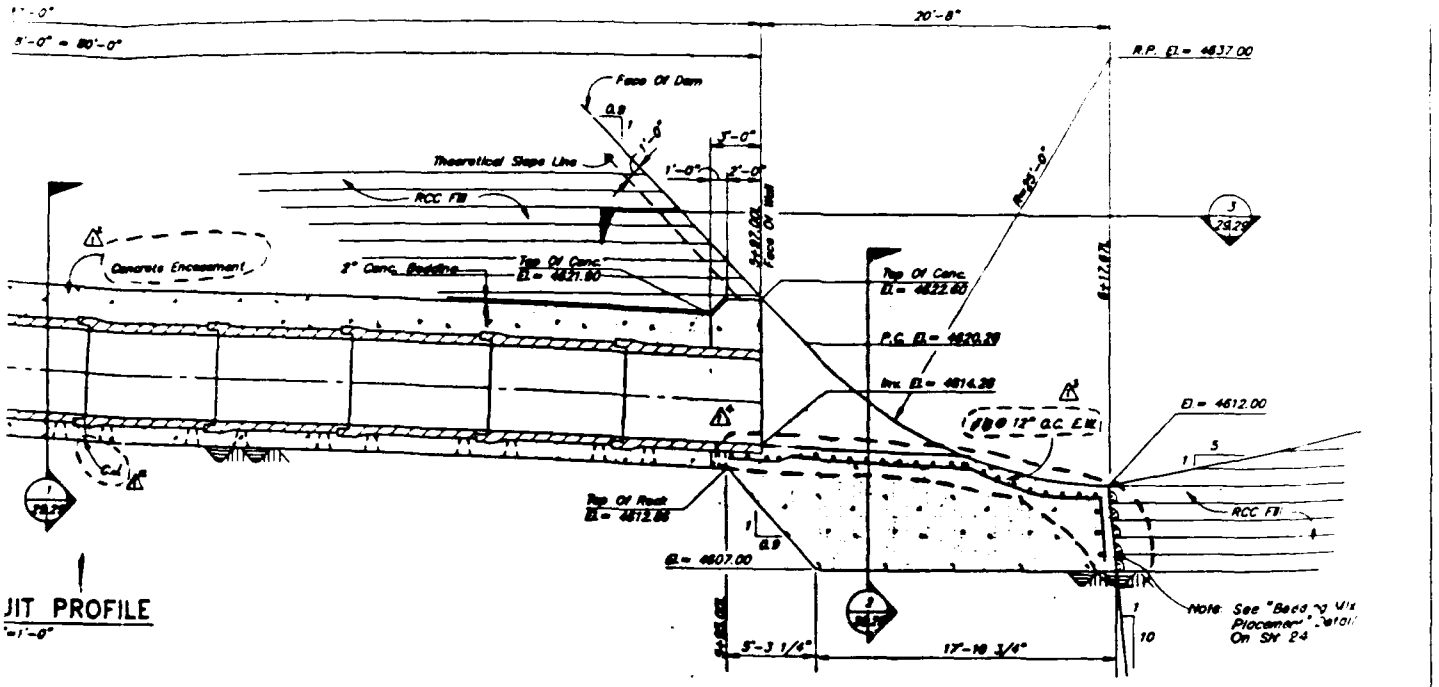


SECTION

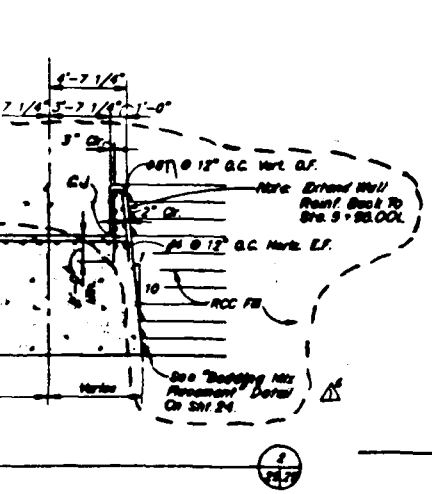
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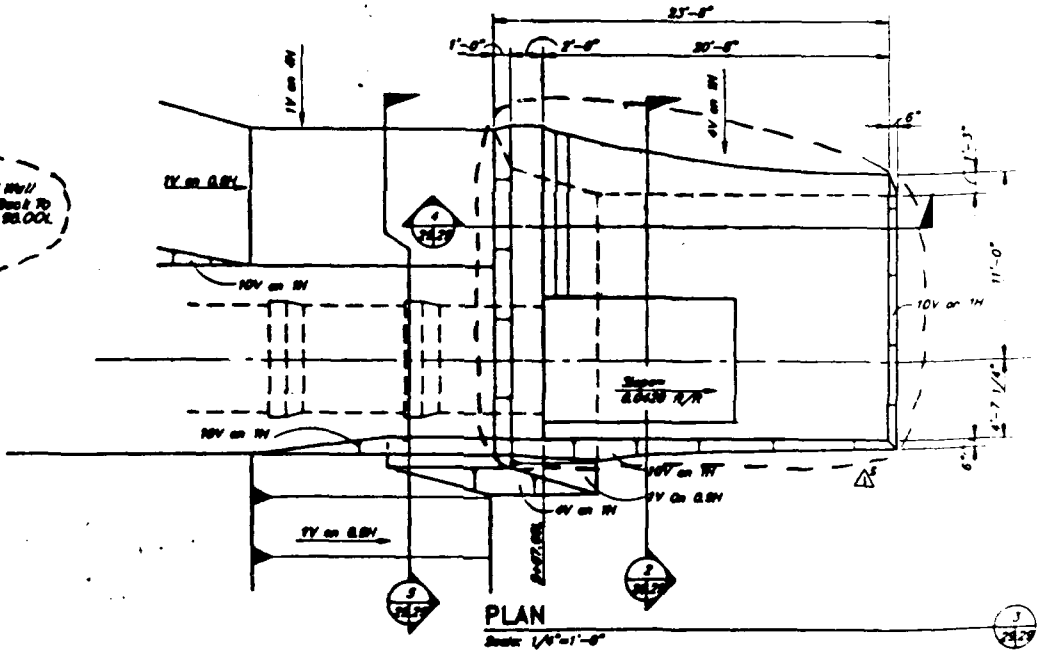
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24.79

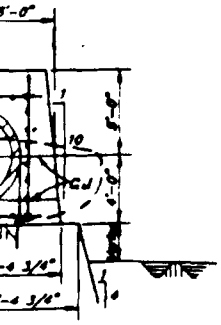


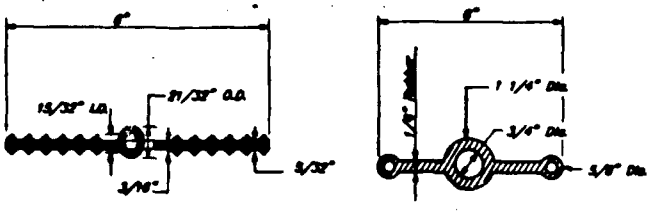
JIT PROFILE





PLAN





POLYVINYL CHLORIDE (PVC)

RUBBER

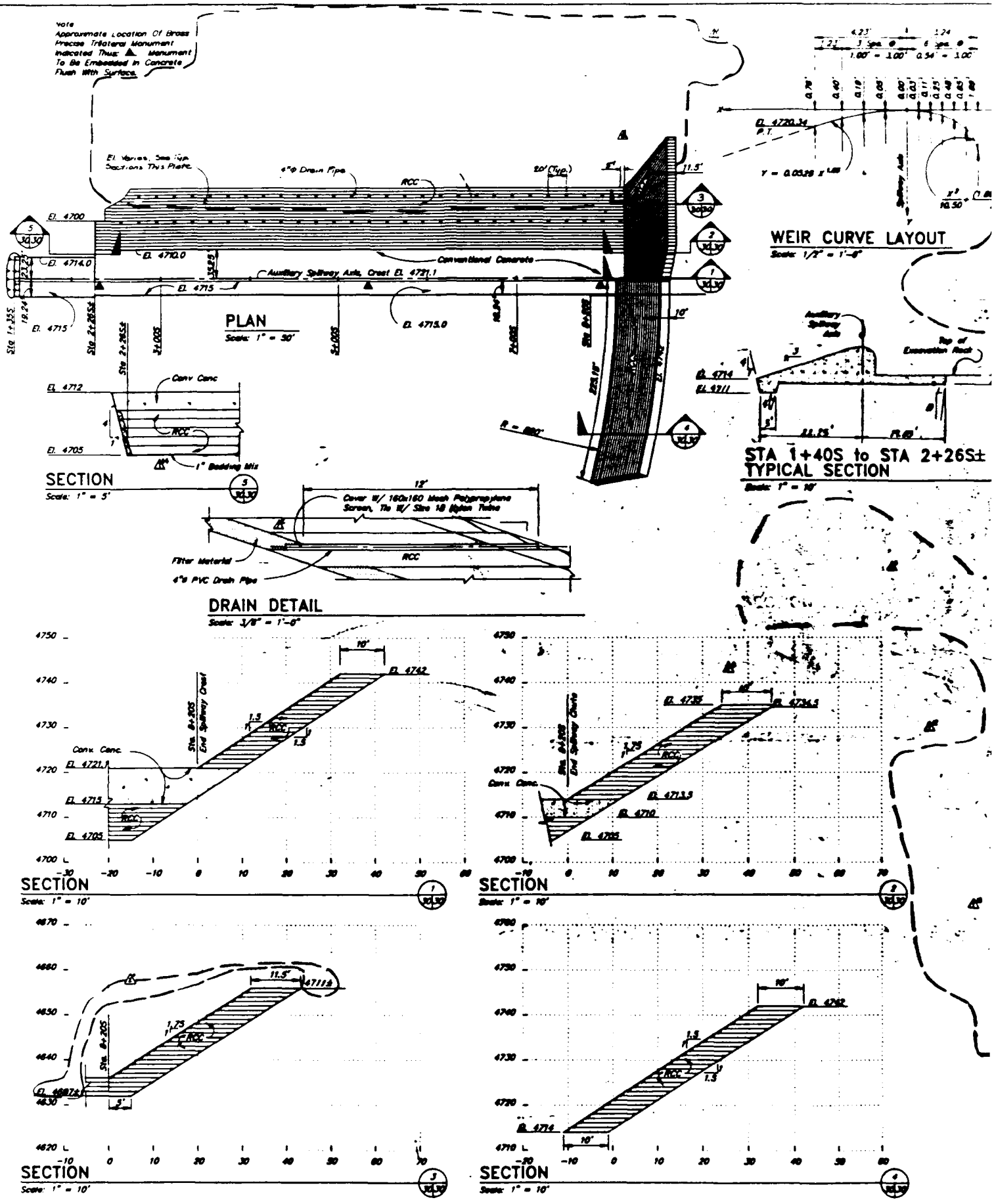
WATERSTOP DETAIL

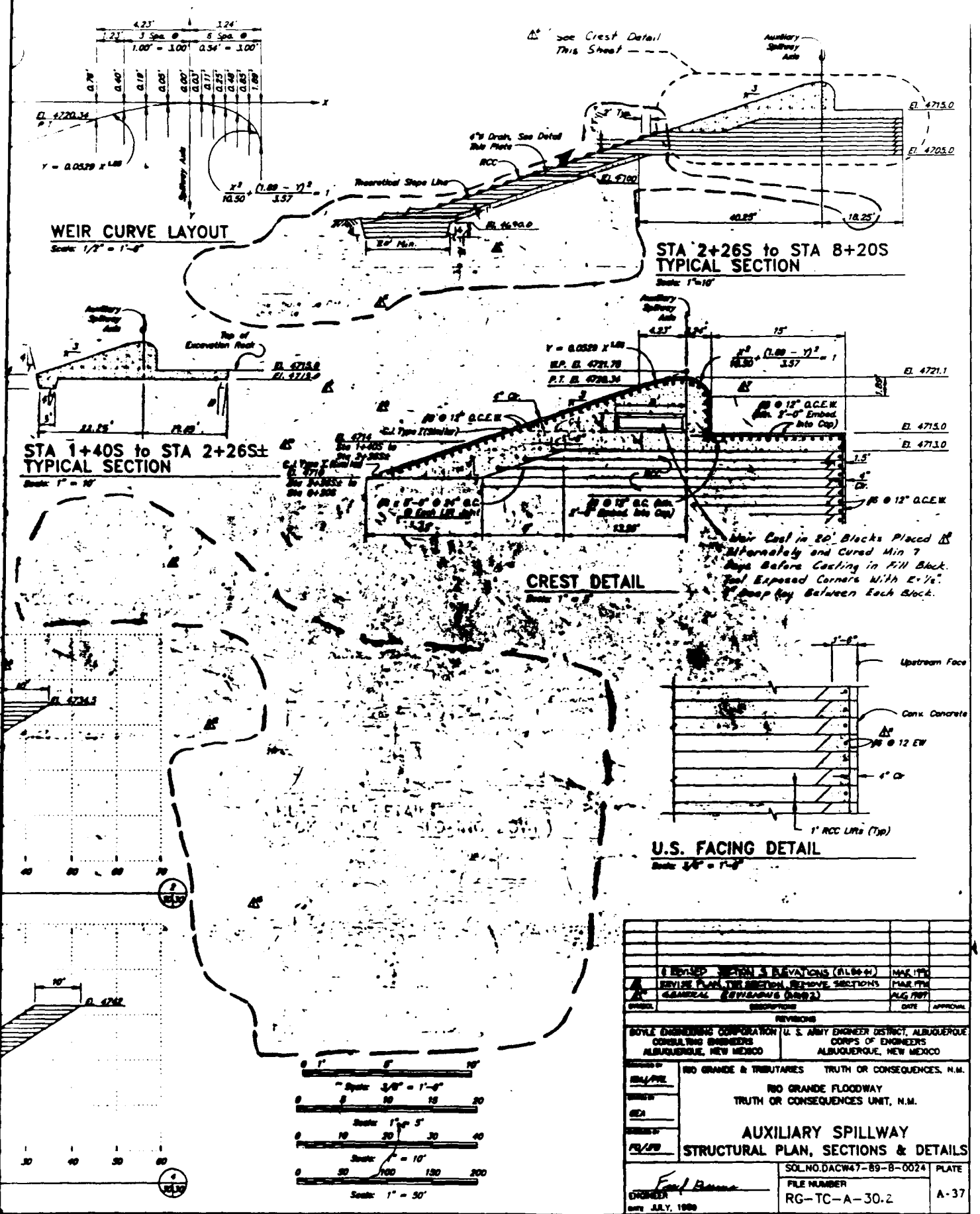
N.T.S.

0 4' 8' 12' 16'

1/4"=1'-0"

	DRAWN	DESCRIPTION
		REVISIONS
BOYLE ENGINEERING CORPORATION CONSULTING ENGINEERS ALBUQUERQUE, NEW MEXICO	U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE CORPS OF ENGINEERS ALBUQUERQUE, NEW MEXICO	
TRP/FB	RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.	
RJH	RIO GRANDE FLOODWAY TRUTH OR CONSEQUENCES UNIT, N.M.	
FB/TRP	LOW LEVEL OUTLET WORKS CONDUIT	
	STRUCTURAL PROFILE, SECTIONS & DETAIL	
Fred Boone ENGINEER DATE JULY, 1989	SOL.NO.DACW47-89-B-0024 FILE NUMBER RG-TC-A-29	PLATE A-36


Note
Approximate Location Of Brass
Precise Triaterra Monument
Indicated Thus: ▲ Monument
To Be Embedded In Concrete
Flush With Surface
El. Varies, See Typ. Sections This Plate
4"ϕ Drain Pipe
RCC
20' (Typ.)
11.5'
El. 4700
El. 4714.0
El. 4712.0
Conventional Concrete
Auxiliary Spillway Axis, Crest El. 4721.1
El. 4715
El. 4715.0
10'
PLAN
Scale: 1" = 50'
El. 4712
Conv Conc
RCC
El. 4705
1" Bedding Mix
SECTION
Scale: 1" = 5'
12'
Cover W/ 160x160 Mesh Polypropylene
Screen, Tie W/ Size 18 Nylon Twine
Filter Material
4"ϕ PVC Drain Pipe
DRAIN DETAIL
Scale: 3/8" = 1'-0"
WEIR CURVE LAYOUT
Scale: 1/2" = 1'-0"
Auxiliary Spillway Axis
Top of Excavation Rock
El. 4714
El. 4711
STA 1+40S to STA 2+26S±
TYPICAL SECTION
Scale: 1" = 10'
Conv. Conc.
El. 4721.1
El. 4715
El. 4705
El. 4742
El. 4735
El. 4734.5
El. 4713.5
El. 4710
El. 4711.5
El. 4714
SECTION
Scale: 1" = 10'



	(REVISED SECTION & ELEVATIONS (EL 84-4))	MAR 1984	
	(REVISE PLAN, TIE SECTION, REMOVE SECTIONS	MAR 1984	
	(GENERAL (ENGRAVING (1982))	AUG 1982	
SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

BOYLE ENGINEERING CORPORATION
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

RIO GRANDE & TRIBUTARIES TRUTH OR CONSEQUENCES, N.M.

RIO GRANDE FLOODWAY
TRUTH OR CONSEQUENCES UNIT, N.M.

AUXILIARY SPILLWAY
STRUCTURAL PLAN, SECTIONS & DETAILS

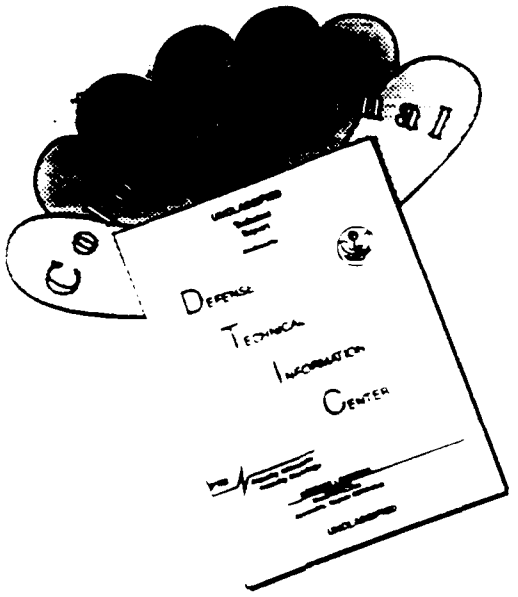
DESIGNED BY: EBA/PRE
DRAWN BY: EBA
CHECKED BY: PG/EB

ENGINEER
DATE: JULY, 1989

SOL.NO.DACW47-89-B-0024
FILE NUMBER RG-TC-A-30.2

PLATE A-37

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